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FLEXIBLE PAVEMENT DESIGN FOR AIRFIELDS (ELASTIC LAYERED METHOD)

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FLEXIBLE PAVEMENT DESIGN FOR AIRFIELDS (ELASTIC LAYERED METHOD)

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CHAPTER 1
INTRODUCTION

1-1. Purpose and Scope.

This manual provides methodology for structural design of flexible airfield pavements by use of layered elastic theory. The procedure is considered to be an alternate to the procedure contained in TM 5-825-2/AFM 88-6, Chap. 2, and because of limited experience in its use, coordination will be made with HQDA (CEMP-ES) Washington, DC 20314 or Headquarters, Air Force Engineering and Service Center on the use of the procedure. Emphasis will be placed on obtaining feedback for verification or modification of the procedure. Contained in the manual are the criteria, material characterization procedures, and other special information necessary for the design of conventional flexible pavements, bituminous concrete pavements, and chemically stabilized pavements. It is anticipated that the use of the procedure will prove beneficial on large projects where cost savings or better performing pavements can be realized by a more rational approach to consideration of all variables.

1-2. Related publications.

A better understanding of the design methodology can be obtained through the study of several related publications which provide background on the development of the methodology and the theory on which the methodology is based. These publications are listed in the Bibliography.

1-3. Design principles.

The structural deterioration of a flexible pavement caused by traffic is normally evidenced by cracking of the bituminous surface course and development of ruts in the wheel paths. The design procedure handles these two modes of structural deterioration through limiting values of the strain at the bottom of the bituminous concrete and at the top of the subgrade. Use of a cumulative damage concept permits the rational handling of variations in the bituminous concrete properties and subgrade strength caused by cyclic climatic conditions. The strain criteria were developed at the U.S. Army Engineer Waterways Experiment Station (WES) and were reported in related publications listed in the Bibliography. The strains used for entering the criteria are computed by the use of Burmister's solution for multilayered elastic continua. The solution of Burmister's equations for most pavement systems will require the use of computer programs and the characterization of the pavement materials by the elastic constants of the modulus of elasticity and Poisson's ratio.

1-4. Pavement response models.

The two computer codes recommended for computing the pavement responses are the CHEVRON code and the BISAR code. When the codes are used, the following assumptions are made:

a. The pavement is a multilayered structure, and each layer is represented by a modulus of elasticity and Poisson's ratio.

b. The interface between layers is continuous; i.e., the friction resistance between layers is greater than the developed shear force.

c. The bottom layer is of infinite thickness.

d. All loads are static, circular, and uniform over the contact area.

Both codes are available at the WES (P. O. Box 631, Vicksburg, MS 39180) and can be furnished to other Army agencies, or arrangements can be made for other agencies to use the code through the computer facilities at the WES.

CHAPTER 2
PRELIMINARY DESIGN DATA

2-1. Climatic factors.

In the design system, two climatic factors—temperature and moisture—are considered to influence the structural behavior of the pavement. Temperature influences the stiffness and fatigue of bituminous materials and is the major factor in frost penetration. Moisture conditions influence the stiffness and strength of the base course, subbase course, and subgrade.

a. Determination of design pavement temperature. The design procedure requires the determination of one design pavement temperature for consideration of vertical compressive strain at the top of the subgrade and horizontal tensile strain at the bottom of cement- or lime-stabilized layers and a different design pavement temperature for consideration of the fatigue damage of the bituminous concrete surface. In either case, a design air temperature is used to determine, from figure 2-1, the design pavement temperature. Temperature data for computing the design air temperatures are available from the National Oceanic and Atmospheric Administration (NOAA) "Local Climatological Data Annual Summary with Comparative Data." With respect to subgrade strain and fatigue of cement- and lime-stabilized base or subbase courses, the design air temperature is the average of the average daily mean temperature and the average daily maximum temperature during the traffic period. For consideration of the fatigue damage of bituminous materials, the design air temperature is the average daily mean temperature. Thus, for each traffic period, two

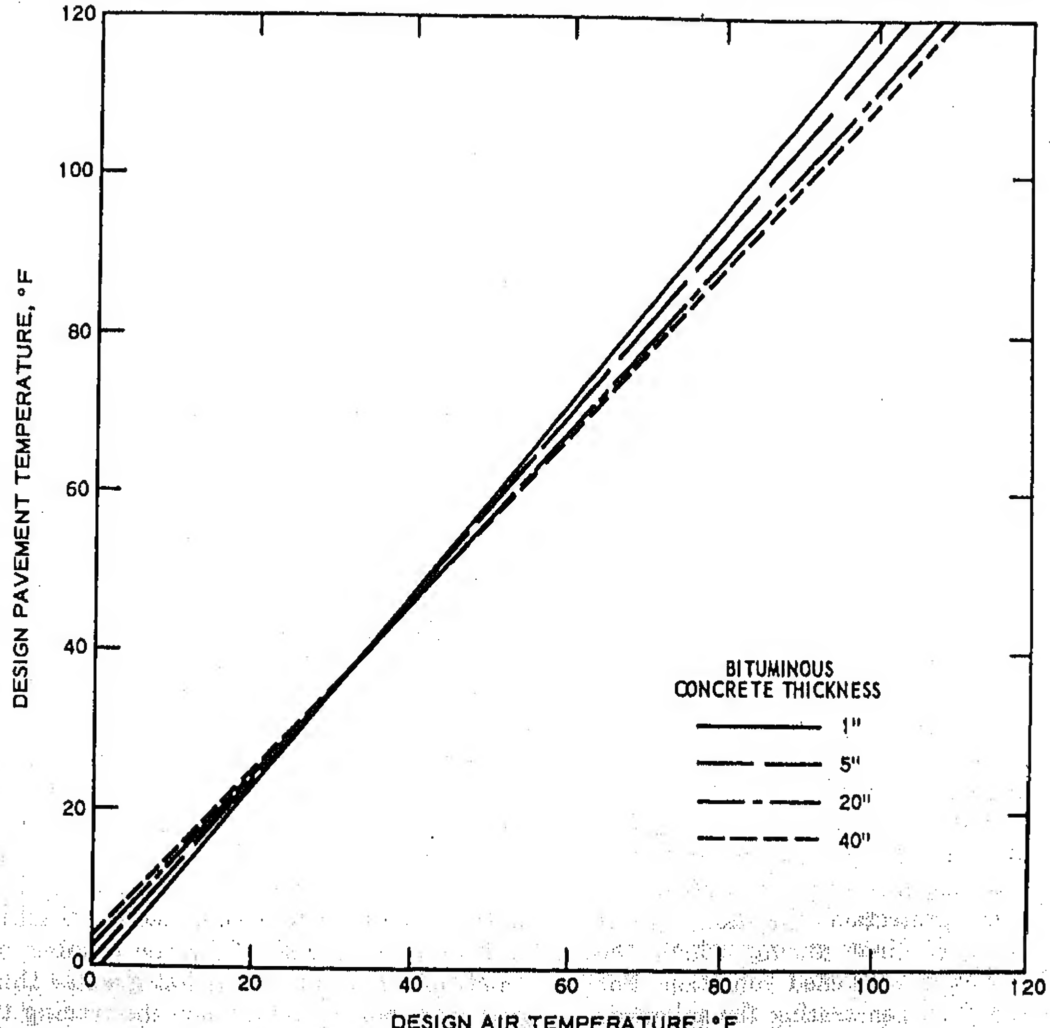

Figure 2-1. Temperature relationships for selected bituminous concrete thickness.

design air temperatures are determined. Normally, monthly traffic periods should be adequate. For design purposes, it is best to use the long-term averages such as the 30-year averages given in the annual summary. The determination of the design pavement temperatures for 10-inch bituminous pavement can be demonstrated by considering the climatological data for Jackson, Mississippi. For the month of August, the average daily mean temperature is 81.5 degrees F. and the average daily maximum is 92.5 degrees F.; therefore, the design air temperature for consideration of the subgrade strain is 87 degrees F., and the design pavement temperature (determined from fig 2-1) would be approximately 100 degrees F. For consideration of bituminous fatigue, the design air temperature for August in Jackson, Mississippi is 81.5 degrees F., resulting in a design pavement temperature of approximately 92 degrees F. These design pavement temperatures are determined for each of the traffic periods. Temperature data for Jackson, Mississippi, (from "Local Climatological Data Annual Summary with Comparative Data, Jackson, Mississippi") are tabulated below:

	Temperature, degrees F.	
Month	*Average Daily Maximum*	*Average Daily Mean*
January	58.4	47.1
February	61.7	49.8
March	68.7	56.1
April	78.2	65.7
May	85.0	72.7
June	91.0	79.4
July	92.7	81.7
August	92.5	81.5
September	88.0	76.0
October	80.1	65.8
November	68.5	55.3
December	60.5	48.9

b. Determination of thaw periods. The effects of temperature on subgrade materials are considered only with regard to frost penetration. The basic requirement of frost protection is given in TM 5-818-2/AFM 88-6, Chap. 4. If the pavement is to be designed for a weakened subgrade condition, i.e., not complete frost protection, the design must consider a period of time during which the subgrade will be in a weakened condition. For designs involving frost penetrating the subgrade the US Army Cold Regionals Research Environmental Laboratory (CRREL) should be contacted for its latest methodology for considering frost effects.

c. Determination of subgrade moisture content for material characterization. In most design situations, pavement design will be predicated on the assumption that the moisture content of the subgrade will approach saturation. If sufficient data are available that indicate the subgrade will not reach saturation, then the design may be based on a lower moisture content. Sufficient data for basing the design on a moisture content lower than saturation would normally consist of field moisture content measurements under similar pavements located in the area. These measurements should be made during the most critical period of the year or when the water table is at its highest elevation. Extreme caution should be exercised when the design is based on other than the saturated condition.

2-2. Traffic data.

The traffic parameters are the designation of the design aircracft, aircraft loading, traffic volume, and traffic area.

a. Traffic volume. The design traffic volume is expressed in terms of total operations of the design aircraft expected during the life of the pavement. This traffic volume must be converted to a number of expected strain repetitions. In converting operations to strain repetitions, the concept of effective gear print is introduced. The effective gear print is the width of pavement that sustains an effective strain repetition at a given depth in the pavement. The effective gear print is a function of the number of tires in a transverse line, the transverse spacing, the width of the contact area, and the effective thickness of pavement above the location of strain. The effective thickness of the pavement is the sum of the thickness of unbound material plus twice the thickness of bound material where a bound material is any asphalt concrete or stabilized layer. Thus, for a pavement having 3 inches of asphalt and 15 inches of unbound gravel, the effective thickness with reference to the strain at the top of the subgrade would be 15 + (2 • 3), or 21 inches, and with respect to the strain at the bottom of the asphalt, the effective thickness would be 2 • 3, or 6 inches. With the determination of the effective thickness, the gear print is computed as illustrated in figures 2-2 and 2-3. If the gear is composed of tracking tires such as tandem gear, then the number of strain repetitions may be somewhat greater than if the gear were not tandem. when the tracing tires are located far enough apart, two distinct strain pulses will occur and the multiplication factor for the

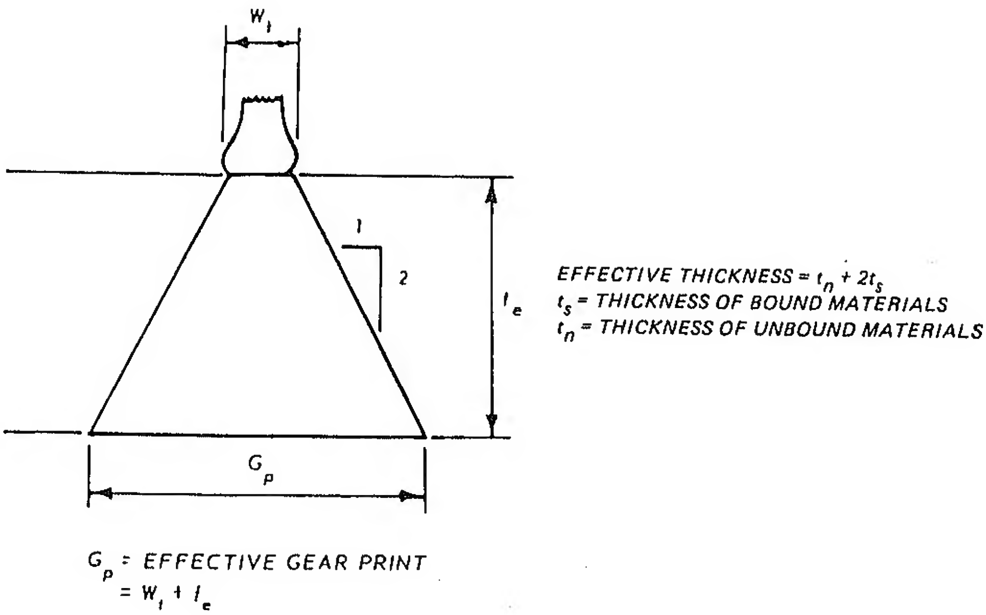

Figure 2-2. Computation of effective gear print for single gear.

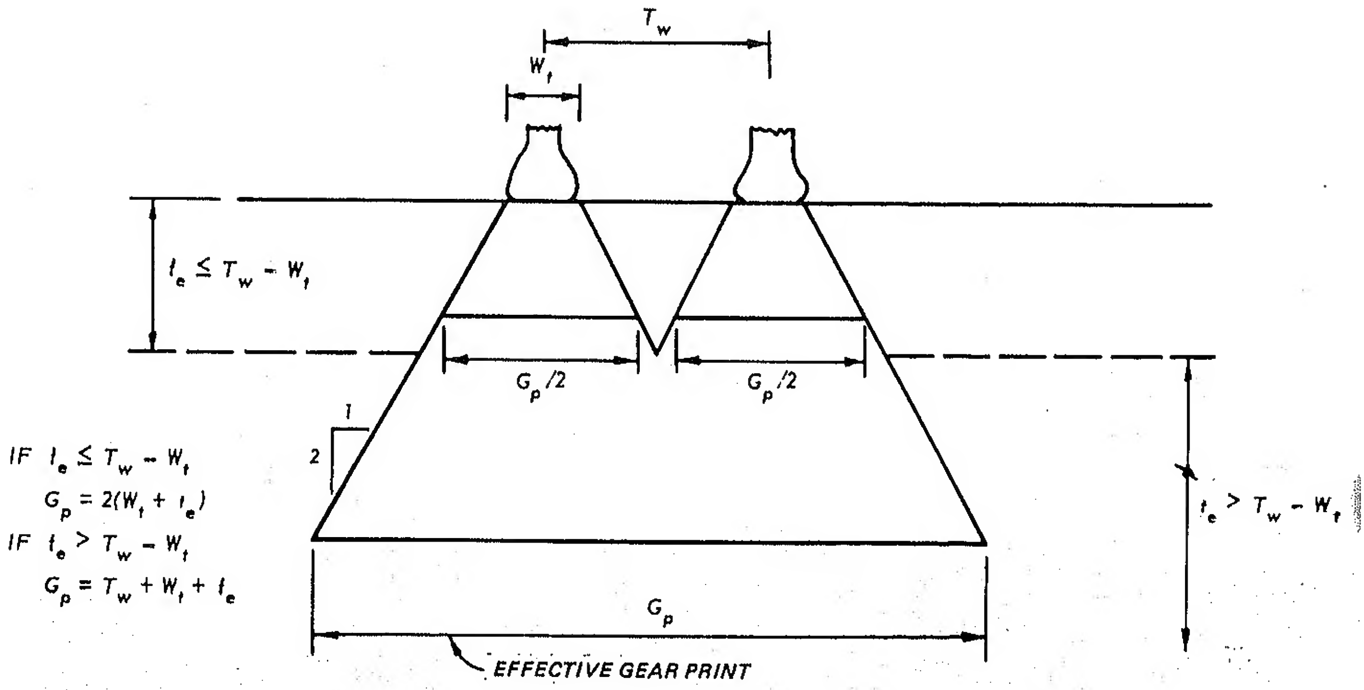

Figure 2-3. Computation of effective gear print for twin gear.

tandem gear is 2; when the tires are sufficiently close, the strain pulses merge into a single pulse and the multiplication factor is 1. The computation of the multiplication factor (F) is shown in figure 2-4. In the figure, B is the spacing between tandem gear; t_e is the effective pavement thickness; and T_w is the radius of a circle having the same area as the contact area. When t_e is less than $B - T_w$, the multiplication factor is 2; when t_e is greater than twice the difference between B and T_w, the multiplication factor is 1. For values of t_e between the two conditions, the multiplication factor (F) is computed based on the equation:

$$F = \frac{3 \cdot (B - T_w) - t_e}{B - T_w} \qquad \text{(eq 2-1)}$$

(1) The concept for conversion of aircraft operations to effective strain repetitions involves assuming that traffic distribution on the pavement can be represented by a normal distribution. For taxiway traffic, the distribution has been shown to have a wander width of approximately 70 inches, and runway traffic has been shown to have a wander width of approximately 140 inches. The Air Force also uses a wander width of 70 inches for the first 1000 feet of runway ends. (*Note* that wander width is defined as the width that contains 75 percent of the applied traffic.) From the normal distribution, the fraction of traffic for which the effective gear print will encompass a given point in the pavement can be computed. This fraction times the multiplication factor (F) gives the number or

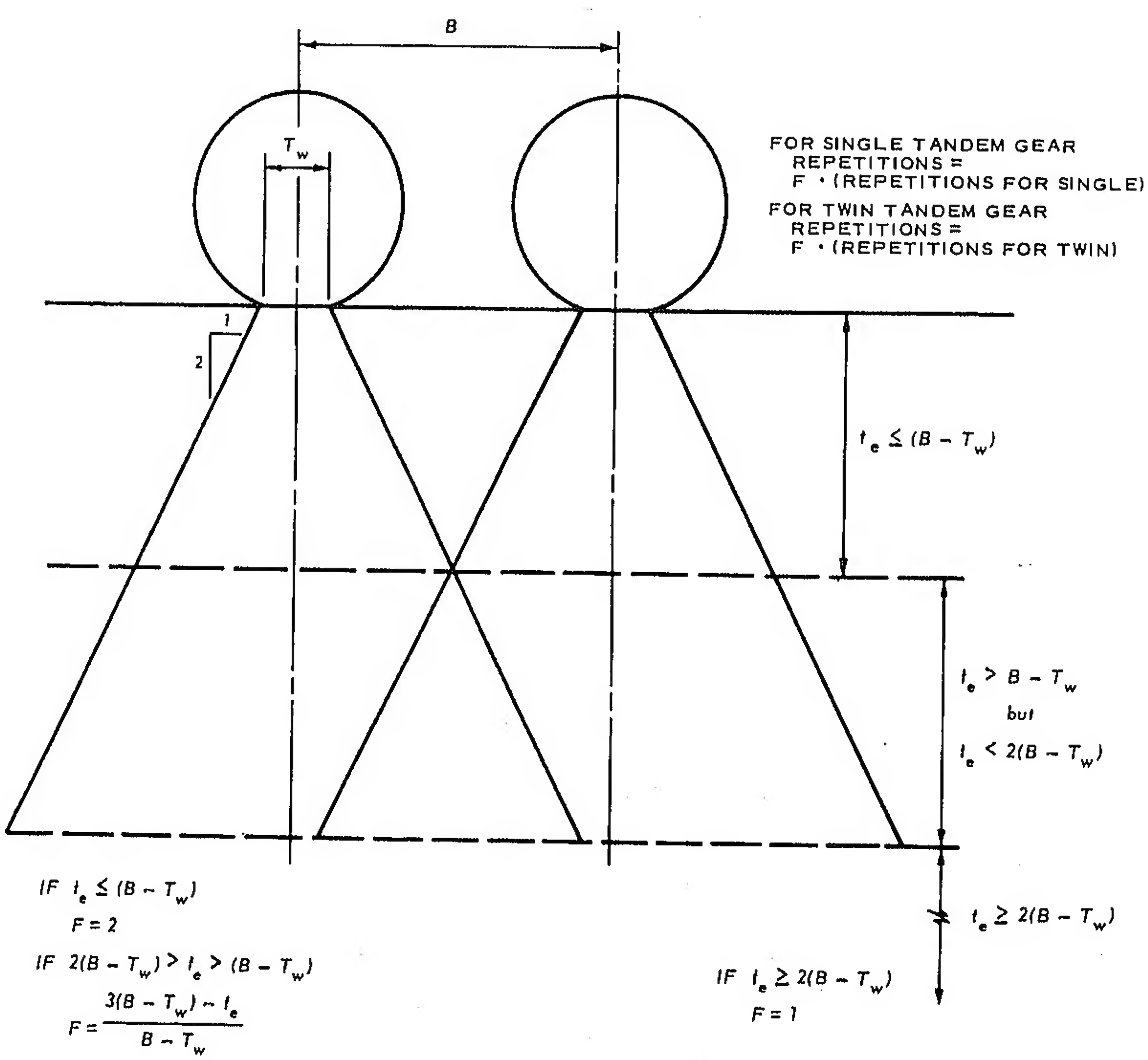

Figure 2-4. Computation of repetition factor for tandem gear.

fraction of the effective strain repetitions at a point in the pavement for each aircraft operation.

(2) The number of effective strain repetitions the pavement sustains at a point for every aircraft operation is the pass-to-strain conversion percent. For an effective thickness of 0 inches, the percentage is the inverse of the pass-to-coverage ratio multiplied by 100. The procedure for computing the pass-to-strain conversion percentage has been computerized, and the factors can easily be computed for single, twin, tandem, twin-twin, and twin-tandem gears, or other gears.

(3) The distribution of the pass-to-strain conversion percentages as a function of point location and effective thicknesses are given in chapter 4. Distributions are given for both runway/non-channelized and taxiway/channelized where the runway/nonchannelized percentage factors are based on a wander width of 140 inches and the taxiway/channelized factors are based on a wander width of 70 inches. The Air Force also uses a wander width of 70 inches for the first 1000 feet of runway ends. These pass-to-strain conversion percentages can be used to convert, for any point location, the number of aircraft operations to effective strain repetitions.

b. Aircraft loading. The aircraft loading and gear characteristics are used in the response model for computing the magnitude of strain. The information needed includes the number of tires, tire spacing, load per tire, and contact pressure. The CHEVRON computer program accepts the load per tire and contact pressure directly; whereas, the BISAR program requires the load per tire and radius of the loaded area. The radius of the loaded area is computed based on the assumption of a uniformly loaded circular area, i.e.,

$$r = \sqrt{\frac{L}{\pi p}} \qquad \text{(eq 2-2)}$$

where

r = radius of loaded area

L = load per tire

p = contact pressure

Note: units should be consistent with units of the section parameters.

In principle, all main tires should be used in computing the strain, but actually only the tires of a single gear need to be used. The distance between gears is sufficiently great to prevent interaction between gears. Within a main gear, some searching for the maximum strain may be needed. For most cases the maximum strain will occur under one of the tires, but for closely spaced tires or strains at a great depth, the maximum may move toward the center of the tire group.

c. Traffic grouping. The traffic is grouped so that within each group each individual pass of an aircraft will cause damage similar to any other pass of the group. That is, the pattern of strain of every pass of the group would be almost the same; then the value of the allowable number of passes (N) would be the same. For this to be true, the loading characteristics for aircraft within a group must be similar, and the single set of material properties must be applicable for all passes within the group. Grouping reduces considerably the design effort, and it is advantageous to reduce traffic to as few groups as possible. Grouping of the aircraft by similar pass-to-strain conversion percents has already been accomplished in chapter 4. Additional subgrouping would be necessary to account for other differences, such as load magnitude and tire pressure. Also, other groupings may be necessary to account for changes in material properties such as changes in subgrade modulus caused by thaw and changes in asphalt modulus caused by temperature. For pavements that are relatively unaffected by changes in temperature and are designed based on a single critical aircraft, it may be possible to reduce the aircraft operations to a single group. In this case, the design procedure simplifies to determining allowable strains for the design of strain repetition and to adjusting the pavement thicknesses to obtain the allowable strain. Where the grouping cannot be reduced to a single group, then the concept of the cumulative damage must be used in the design process.

2-3. Material characterization.

Characterization of the pavement materials requires the quantification of the material stiffness as defined by the resilient modulus of elasticity and Poisson's ratio and, for selected pavement components, a fatigue strength as defined by a failure criterion. Inasmuch as possible, repeated load laboratory tests designed to simulate aircraft loading are used to determine the resilient stiffness of the materials

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particular, the gradation, strength, and durability requirements as stated in TM 5-825-2/AFM 88-6, Chap. 2 must be maintained.

a. Modulus of elasticity.

(1) *Bituminous mixtures.* The term "bituminous mixtures" refers to a compacted mixture of bitumen and aggregate designed in accordance with standard practice. The modulus for these materials is determined by use of the repetitive triaxial tests. The procedure for preparation of the sample is given in chapter 8 with the procedure for the conduct of the repetitive triaxial test given in chapter 9.

(*a*) The stiffness of the bituminous mixtures will be greatly affected by both the rate of loading and by temperature. For runway design, a loading rate of 10 hertz is recommended. For taxiway design and apron, a loading rate of 2 hertz is suggested. These loading rates are appropriate for aircraft speeds of over 100 miles per hour on runways and less than 20 miles per hour on taxiways and aprons. Specimens should be tested at temperatures of 40, 70, and 100 degrees F. so that a modulus-temperature relationship can be established. If temperature data indicate greater extremes than 40 and 100 degrees F., tests should be conducted at these extreme ranges if possible. The modulus value to be used for each strain computation would be the value applicable for the specific pavement temperature determined from the climatic data.

(*b*) An indirect method of obtaining an estimated modulus value for bituminous concrete is presented in detail in chapter 10. Use of this method requires that the ring-and-ball softening point and the penetration of the bitumen as well as the volume concentration of the aggregate and percent air voids of the compacted mixture be determined.

(2) *Unbound granular base and subbase course materials.* The terms "unbound granular base course material" and "unbound granular subbase course material" as used herein refer to materials meeting grading requirements and other requirements for base and subbase for airfield pavements, respectively. These materials are characterized by use of a chart in which the modulus is a function of the underlying layer and the layer thickness. The chart and the procedure for use of the chart are given in chapter 6.

(3) *Stabilized material.* The term "stabilized material" as used herein refers to soil treated with such agents as bitumen, portland cement, slaked or hydrated lime, and fly ash or a combination of such agents to obtain a substantial increase in the strength of the material over the materials untreated natural strength. Stabilization with portland cement, lime, fly ash or other agent that causes a chemical cementation to occur shall be referred to as chemical stabilization. Chemically treated soils having unconfined compressive strengths greater than the minimum strength specified for subbases are considered to be stabilized materials and should be tested in accordance with the methods specified for stabilized materials. Chemically treated soils having unconfined compressive strengths less than that specified for subbases are considered to be modified subgrade soils and should be tested under the provisions for subgrade soils. Most likely this will result in using the maximum allowable subgrade modulus. Bituminous-stabilized materials should be characterized in the same manner as bituminous concrete. Stabilized materials other than bituminous-stabilized should be characterized using flexural beam tests or cracked section criteria. Flexural modulus values determined directly from laboratory tests can be used when the effect of cracking is not significant and the computed strain based on this modulus does not exceed the allowable strain for the material being used.

(*a*) The general approach in the flexural beam test is to subject the specimen to repeated loadings at third points, measure the maximum deflection at the center of the beam (i.e., at the midpoint of the neutral axis), and calculate the values for the flexural modulus based on the theory of a simply supported beam. A correlation factor for stress is applied.

(*b*) Procedures for preparing specimens of and conducting flexural beam tests on chemically stabilized soils are presented in detail in chapter 7.

(*c*) The stabilized material for the base and subbase must meet the strength and durability requirement of TM 5-822-4/AFM 88-7, Chap. 4/NAVFAC DM 21.5. Basically, the strength requirements are as summarized in table 2-1.

(*d*) It should be recognized that lime-stabilized materials will continue to gain strength with time; therefore, if sufficient evidence is available that indicates a lime-stabilized material will acquire adequate strength prior to traffic, then the 28-day strength requirement may be waived.

(4) *Subgrade soils.* The term "subgrade" as used herein refers to the natural, processed, or fill soil foundation not meeting the requirements for a base or subbase on which a pavement structure is placed. The modulus of the subgrade is determined through the use of the repetitive triaxial test. For

Table 2-1. **Minimum unconfined compressive strengths for cement, lime, and combined lime-cement-fly ash stabilized soils**

Stabilized Layer	Unconfined Compressive Strength, psi [a] For Cited Design Aircraft Loading, kips		
	<30	30 to 200	>200
Base course	500	1000	1000
Subbase course	250	500	500

[a] Unconfined compressive strength determined at 7 days for cement stabilization and 28 days for lime or lime-cement-fly ash stabilization.

most subgrade soils, the modulus is greatly affected by changes in moisture content and state of stress. In normal airport construction, the subgrade soil is compacted to 95 to 100 percent of modified American Association of State Highway and Transportation Officials (AASHTO) maximum density and at or near the optimum moisture content for that compaction effort. As a result of normal moisture migration, water table fluctuation, and other factors, the moisture content of the subgrade soil can increase and approach saturation with only a slight change in density. Since the strength and stiffness of fine-grained materials are particularly affected by such an increase in moisture content, these soils should preferably be tested in the near-saturation state. Two methods are available to obtain a specimen with this moisture content: the soil can be either molded at optimum moisture content and subsequently saturated or molded at the higher moisture content using static compaction methods. Although evidence exists that the resilient properties of both specimen types are similar, most of the tests reported in the literature involve materials compacted using the standard AASHTO compaction effort. It is not apparent whether this concept is valid for materials compacted at the higher densities; therefore, for the test procedures presented herein, back-pressure saturation of samples compacted at optimum is recommended for developing high moisture contents in test specimens.

(*a*) For cohesive subgrades, the resilient modulus of the subgrade will normally decrease with an increase in deviator stress, and therefore, the modulus is determined as a function of deviator stress. The modulus of granular subgrades will be a function of the first invariant. Procedures for specimen preparation, testing, and interpretation of test results for cohesive and granular subgrades are presented in chapter 5. For the layered elastic theory design procedure, however, the maximum allowable modulus for a subgrade soil should be restricted to 30,000 pounds per square inch (psi).

(*b*) In areas where the subgrade is to be subjected to freeze-thaw cycles, the subgrade modulus must be determined during the thaw-weakened state. Testing soils subject to freeze-thaw requires specialized test apparatus and procedures. CRREL should be contacted before attempting characterization of subgrade soils subjected to freeze-thaw.

(*c*) For some design situations, estimating the resilient modulus of the subgrade (M_R) based on available information may be necessary when conducting the repetitive load triaxial tests. An estimate of the resilient modulus can be made from the relationship of $M_R = 1500 \cdot CBR$, where CBR is the California Bearing Ratio. The relationship does provide a method for checking the reasonableness of the laboratory results.

b. Poisson's ratio. Because of the complexity of laboratory procedures involved in the direct determination of Poisson's ratio for pavement materials and because of the relatively minor influence on pavement design of this parameter when compared with other parameters, use of values commonly recognized as acceptable is

recommended. These values for the four classes of pavement materials considered herein are presented in the following tabulation:

Pavement Material	*Poisson's Ratio υ*
Bituminous concrete	0.5 for E < 500,000 psi 0.3 for E > 500,000 psi
Unbound granular base or subbase course	0.3
Chemically stabilized base or subbase course	0.2
Subgrade	
Cohesive subgrade	0.4
Cohesionless subgrade	0.3

2-4. Subgrade evaluation.

Sections 2 and 3 of TM 5-825-2, which provide for the evaluation of the subgrade for design by the CBR design procedure, also provide the background for evaluation of the subgrade modulus. Borings will have been taken and soil tests conducted according to Section 2.5 of TM 5-825-2/AFM 88-6, Chap. 2. After the establishment of the grade line, the pavement will be grouped as to soil type, strength, expected moisture content, compaction requirements, and other characteristics. For each soil group, a minimum of six resilient modulus tests should be conducted and the design modulus determined according to procedures given in chapter 7. The design modulus would be the average of the moduli obtained from the testing.

2-5. Design criteria.

The damage factor (DF) is defined as $DF = \frac{n}{N}$, where n is the number of effective strain repetitions and N is the number of allowable strain repetitions. The cumulative damage factor is the sum of the damage factors for all aircraft. The value of n is determined from the number of aircraft operations. The value of N must be determined from the computed strain and the appropriate criteria. Basically, there are three criteria to determine N:

- The allowable number of repetitions as a function of the vertical strain at the top of the subgrade.
- The allowable number or repetitions as a function of the horizontal strain at the bottom of the bituminous concrete.
- The allowable number of repetitions as a function of the horizontal strain at the bottom of a chemically stabilized base or chemically stabilized subbase.

It should be noted that there is no strain criterion for unbound base. In the development of the procedure it has been assumed that unbound base and subbase that meet COE specification for quality will perform satisfactorily.

a. Subgrade strain criteria. The subgrade strain criteria were developed by the WES from the analysis of field test data and present the allowable number of strain repetitions as a function of strain

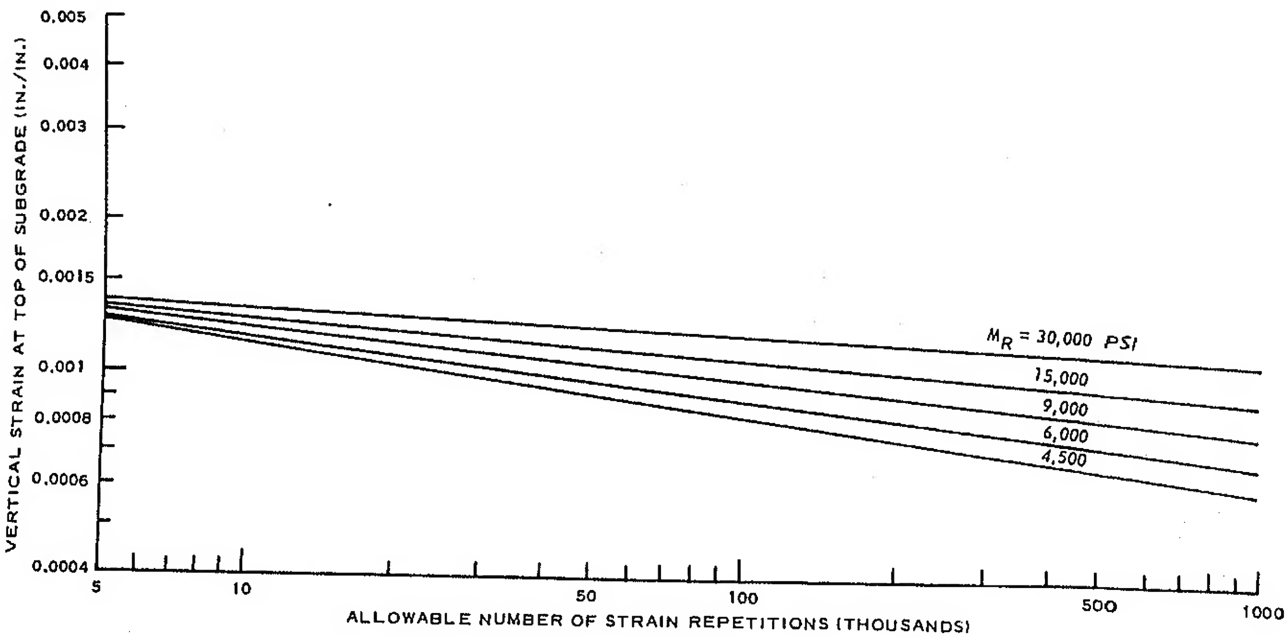


Figure 2-5. Design criteria based on subgrade strain.

magnitude. The data analysis indicated that the relationship between allowable repetitions and strain magnitude is slightly different for subgrades having different resilient moduli. The criteria are presented in graph form in figure 2-5. These criteria can be approximated using the equation

$$\text{allowable repetitions} = 10{,}000 \cdot \left(\frac{A}{S_s}\right)^B \qquad \text{(eq 2-3)}$$

where

$A = 0.000247 = 0.000245 \log M_R$

S_s = vertical strain at the top of the subgrade (in/in)

$B = 0.0658\ M_R^{0.559}$

M_R = resilient modulus of the subgrade psi

b. Asphalt strain criteria.

(1) The primary means recommended for determining values of limiting horizontal tensile strain for bituminous concrete is the use of the repetitive load flexural beam test on laboratory-prepared specimens. Procedures for the test are presented in detail in chapter 11. Several tests are run at different stress levels and different sample temperatures such that the number of load repetitions to fracture can be represented as a function of temperature and initial stress. The initial stress is converted to initial strain to yield criteria based on the tensile strain of the bituminous concrete.

(2) An alternate method for determining values of limiting tensile strain for bituminous concrete is the use of the provisional laboratory fatigue data employed by Heukelom and Klomp. These data are presented in chapter 11 in the form of a relationship between stress, strain, load repetitions, and elastic moduli of bituminous concrete. The data may be approximated by the equation

$$\text{Allowable strain repetitions} = 10^X \qquad \text{(eq 2-4)}$$

where

$X = 5 \log S_A - 2.665 \log E = 2.68$

S_A = tensile strain of asphalt (in/in)

E = elastic modulus of the bituminous concrete (psi)

c. Chemically stabilized layers. For cement- and lime-stabilized materials, the criteria are to be developed using test procedures outlined in chapter 7. When the flexural fatigue tests are not possible, then a preestablished relationship as shown in figure 2-6 should be used.

d. Computer programs for computing cumulative damage factor. Two computer codes, SUBGRAD and ASPHALT, have been written for computing the subgrade and asphalt damage factors based on

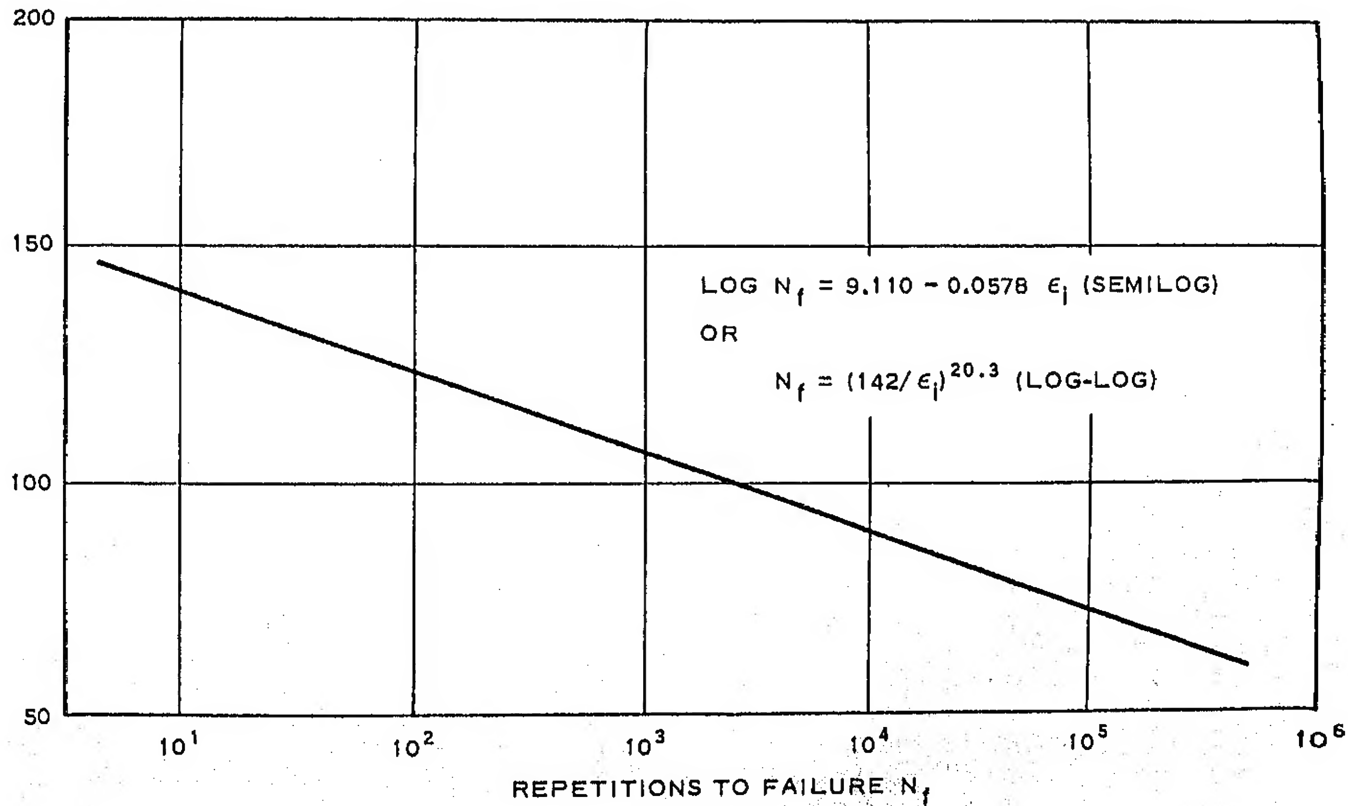


Figure 2-6. Fatigue life of flexural specimens.

equations 2-3 and 2-4. The program SUBGRAD, listed in appendix C, computes the subgrade damage, and the program ASPHALT, listed in appendix D, computes the asphalt damage. Both programs require the input of the material strains obtained by the running of the layered elastic computer programs. The listing of the programs contains an explanation of the input and instructions on the use of the programs. An example use of the programs is given in appendix B.

CHAPTER 3
DETERMINATION OF PAVEMENT THICKNESS

3-1. Conventional flexible pavements.

a. General. Conventional pavements consist of relatively thick aggregate layers with a thin (3- to 5-inch) wearing course of bituminous concrete. In this type of pavement, the bituminous concrete structure is a minor structural element of the pavement, and thus, the temperature effects on the stiffness properties of the bituminous concrete may be neglected. Also, it must be assumed that if the minimum thickness of bituminous concrete is used as specified in TM 5-825-2/AFM 88-6, Chap. 2, then fatigue cracking will not be considered. Thus, for a conventional pavement, the design problem is one of determining the thickness of pavement required to protect the subgrade. The steps for determining the required thickness for nonfrost areas are:

(1) The subgrade resilient modulus is determined based on the soil exploration, climatic conditions, and laboratory testing. The resilient modulus of the bituminous concrete is assumed to be 200,000 psi.

(2) The traffic data determine the design loadings and repetitions of strain.

(3) An initial pavement section is determined from the minimum thickness requirements as determined using TM 5-825-2/AFM 88-6, Chap. 2 or by estimation. The resilient modulus of the base and of the subbase is determined based on the chart and the initial thickness.

(4) The vertical strain at the top of the subgrade is computed for each aircraft being considered in the design.

(5) The number of allowable strain repetitions for each computed strain is determined from the subgrade strain criteria.

(6) The value of n/N is computed for each aircraft and summed to obtain the cumulative damage factor.

(7) The assumed thicknesses are adjusted to make the value of the cumulative damage factor approach 1. This may be accomplished by first making the computations for three thicknesses and developing a plot of thickness versus damage factor. From this plot the thickness that gives a damage factor of 1 may be selected.

b. Frost conditions. Where frost conditions exist and the design is to be based on a base and subbase thickness less than the thickness required for complete frost protection, the design must be based on two traffic periods as described in chapter 2. In some cases, it may be possible to replace part of the subgrade with material not affected by cycles of freeze-thaw but which will not meet the specifications for a base or subbase. In this case, the material must be treated as a subgrade and characterized by the procedures given for subgrade characterization. For information of designing for frost conditions, CRREL should be contacted.

3-2. Bituminous concrete pavements.

The bituminous concrete pavement differs from the conventional flexible pavement in that the bituminous concrete is sufficiently thick to contribute significantly to the strength of the pavement. In this case, the variation in the stiffness of the bituminous concrete caused by yearly climatic variations must be taken into account by dividing the traffic into increments during which variation of the resilient modulus of the bituminous concrete is at a minimum. One procedure is to determine the resilient modulus of the bituminous concrete for each month, then group the months when the bituminous concrete has a similar resilient modulii. Thus, it may be possible to reduce the traffic to three or four groups. Also, since the bituminous concrete is a major structural element, the failure of this element due to fatigue cracking must be checked. The flow diagram for design of the bituminous concrete pavements is given in figure 3-1.

3-3. Pavements with a chemically stabilized base course.

For a pavement having a chemically stabilized base course and aggregate subbase course, damage must be accumulated for subgrade strain, for horizontal tensile strain at the bottom of the bituminous concrete surfacing, and for horizontal tensile strain at the bottom of the chemically stabilized layer. Normally in this type of pavement, the base course resilient modulus is sufficiently high (≥ 100,000 psi) to prevent fatigue cracking of the bituminous concrete surface course (where the bituminous concrete surface course has a thickness equal to or greater than the minimum required of the CBR base course given in table 6-4 of TM 5-825-2/AFM 88-6, Chap. 2), and thus this mode of failure is only a minor consideration. For

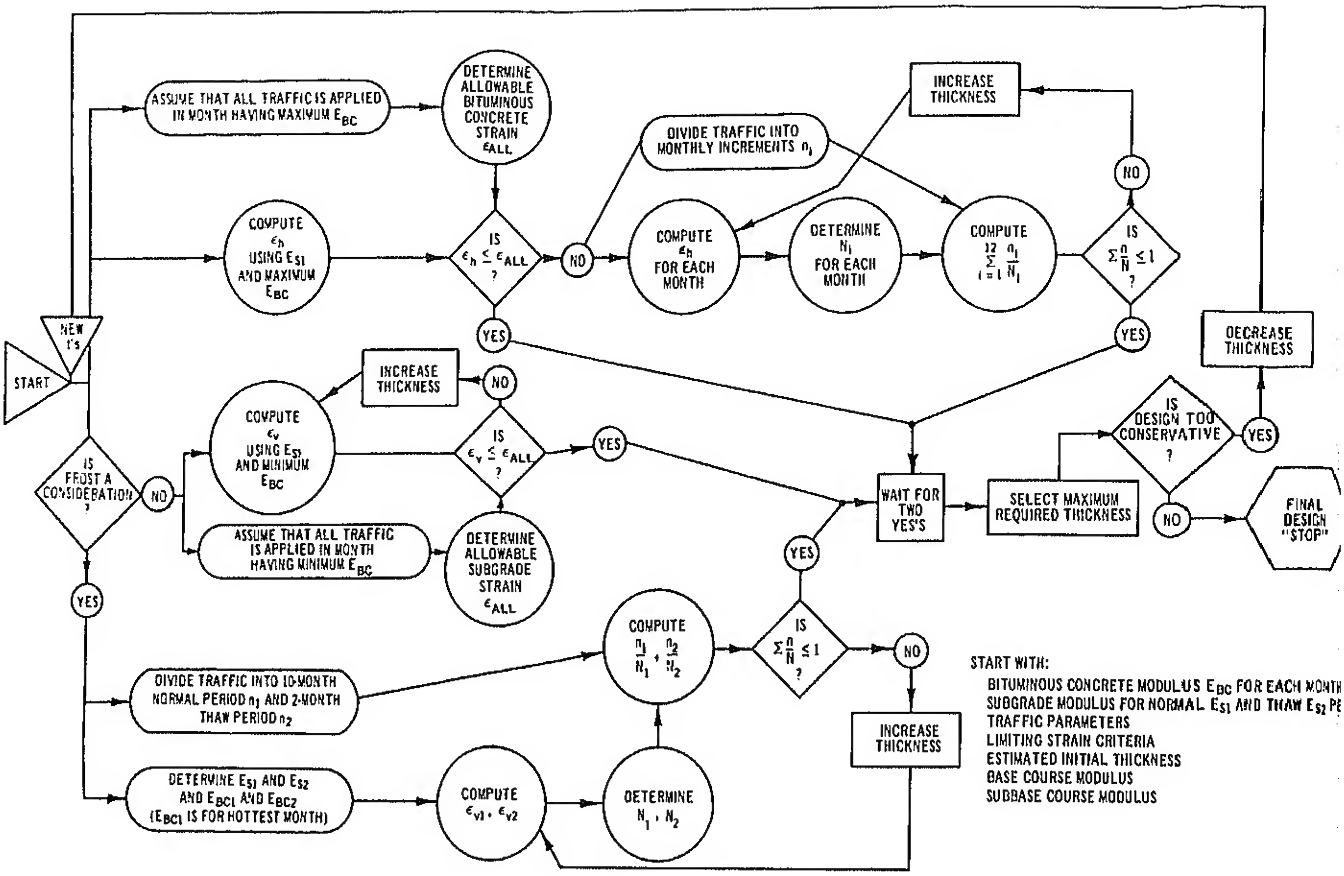


Figure 3-1. Flow diagram of important events of bituminous flexible pavement.

most cases, a very conservative approach can be taken in checking for this mode of failure; i.e., all the traffic can be grouped into the most critical time period and the computed bituminous concrete strain compared with the allowable strain. If the conservative approach indicates that the surface course is unsatisfactory, then the damage should be accumulated in the same manner used for conventional flexible pavement. For the pavement having a stabilized base or subbase, checking the subgrade strain criteria becomes more complicated than for conventional flexible or bituminous concrete pavements. Two cases in particular should be considered. In the first case, the stabilized layer is considered to be continuous, with cracking due only to curing and temperature. In the second case, the stabilized layer is considered cracked because of load. The first step in evaluating the stabilized layer is to compute the horizontal tensile strain at the bottom of the stabilized layer and the vertical compressive strain at the top of the subgrade, under assumptions that the stabilized layer is continuous and has a modulus value as determined by the flexural resilient modulus test. To account for the increase in stress due to loadings near shrinkage cracks, the computed strains should be multiplied by 1.5 for comparison with the allowable strains. If the analysis shows that the stabilized base will not crack under load, then it will be necessary to compare the adjusted value of subgrade strain with the allowable subgrade strain. If this analysis indicates that the adjusted strain is not less than or equal to the allowable strain, then the thickness should be increased and the process repeated, or the section should be checked under the assumption that the base course will crack and behave as a granular material. The cracked stabilized base course is represented by a reduced resilient modulus value, which is determined from the relationship between resilient modulus and unconfined compressive strength shown in figure 3-2. When the cracked base concept is used, only the subgrade criteria need to be satisfied. The section obtained should not differ greatly from the section obtained by use of the equivalency factors in table 3-1. A flow diagram for the design of this type of pavement is shown in figure 3-3.

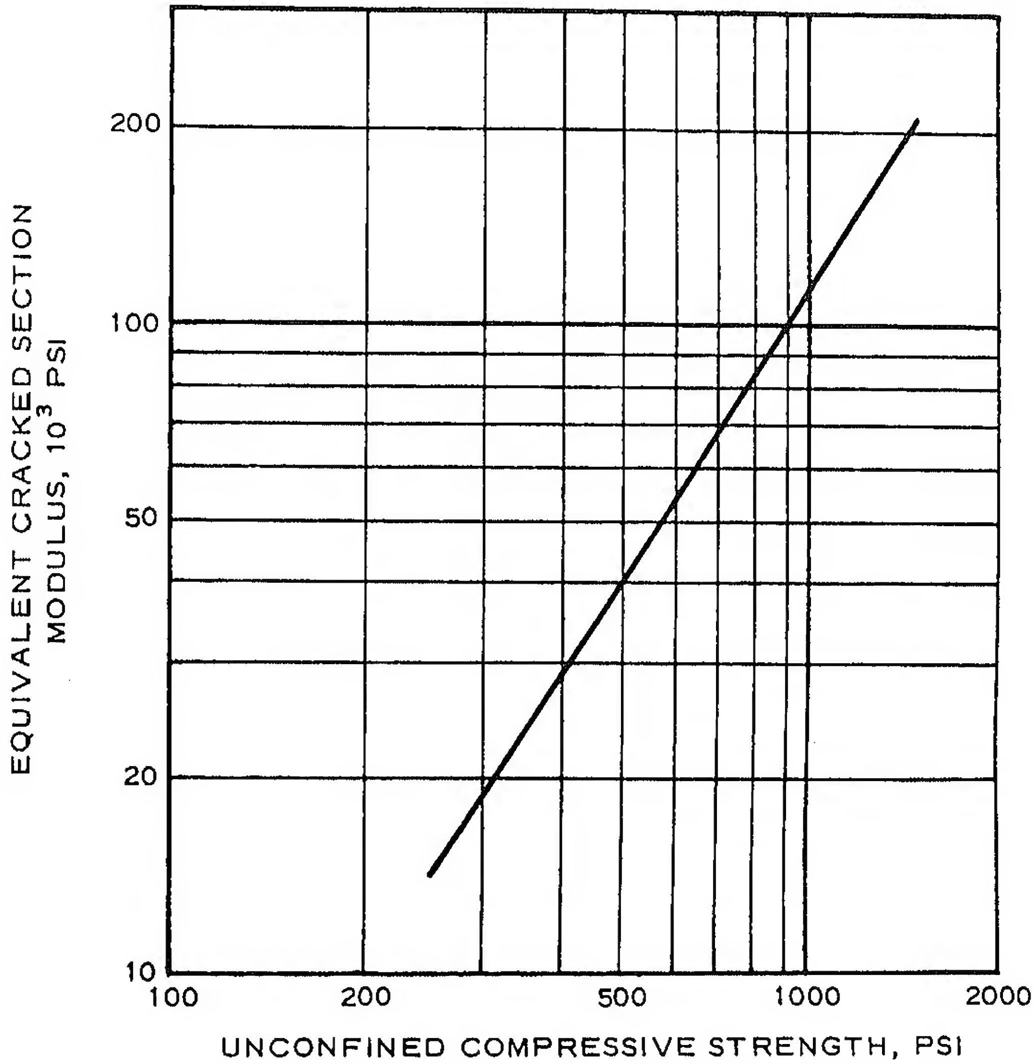


Figure 3-2. Relationship between cracked section modulus and unconfined compressive strength.

3-4. Pavements with stabilized base and chemically stabilized subbase courses.

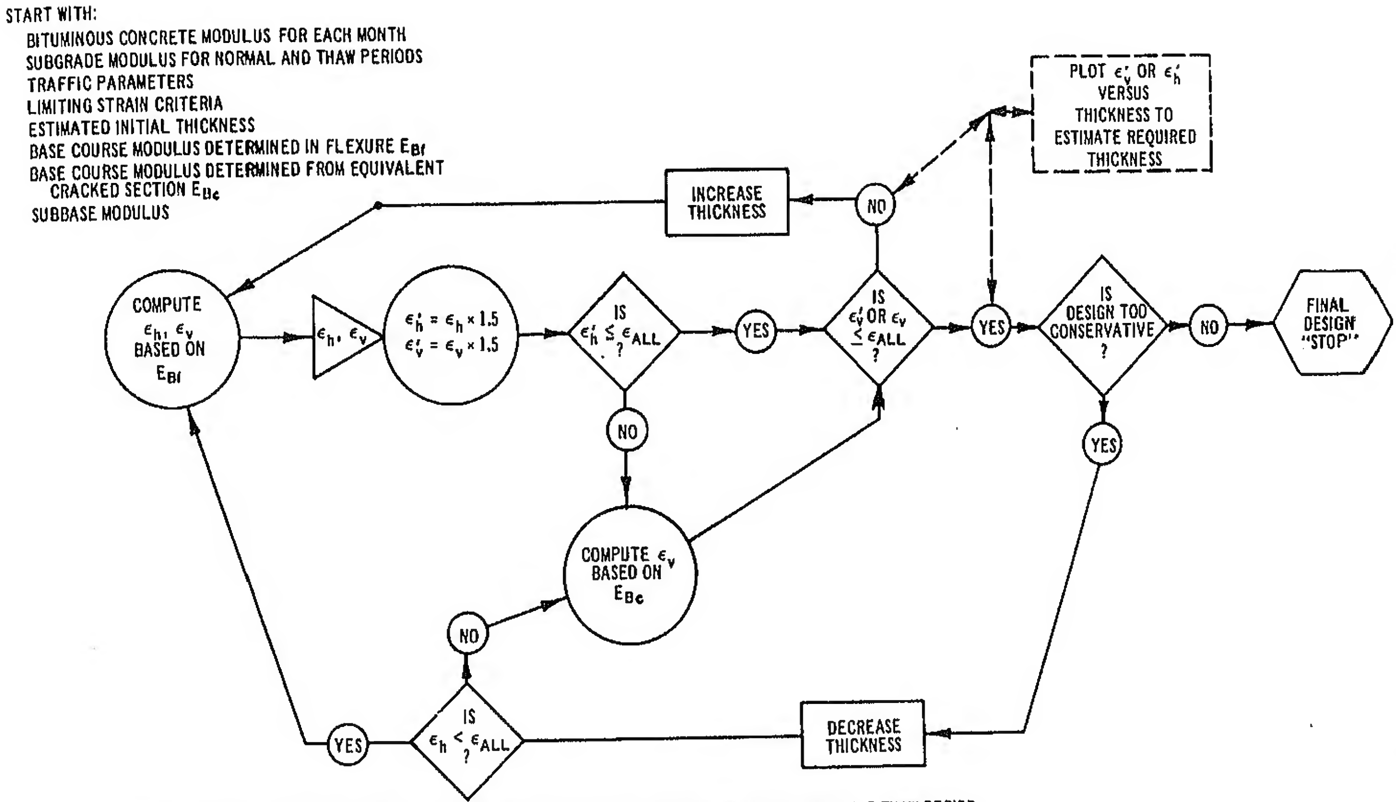
This type of pavement is handled almost identically to a pavement with a stabilized base. If the base is a bituminous-stabilized material, then the cumulative damage procedure must be employed to determine if the subbase will crack. If the analysis indicates that the subbase will crack due to loading, an equivalent cracked section modulus is determined from figure 3-3, and the pavement is treated as a bituminous concrete pavement. If both the base and subbase courses are chemically stabilized, then both layers must be checked for cracking. A conservative approach is taken by checking for cracking of one layer by considering the other stabilized layer as cracked and having a reduced modulus. The vertical compressive strain at the top of the subgrade is computed by use of the flexural modulus or the reduced modulus, as appropriate. If either of the two layers is considered uncracked, then the computed subgrade strain is multiplied by 1.5 to account for the shrinkage cracks that will exist. The basic flow diagram for this type of pavement is shown in figure 3-4.

Table 3-1. Equivalency factors for various materials.

Material	Equivalency Factor[a]
ABC	1.70
Bituminous-stabilized GW, GP, GM, GC, SW, SP, SM, and SC	1.50
Cement-stabilized GW, GP, SW, and SP	1.60
Cement-stabilized GM and GC	1.45
Cement-stabilized ML, MH, CL, and CH	1.25
Cement-stabilized SM and SC	1.15
Lime-stabilized ML, MH, CL, and CH	1.10
Lime-and-fly-ash-stabilized ML, MH, CL, and CH	1.15
Unbound crushed stone base course	1.40
Unbound granular subbase course	1.00

[a] Equivalency factors are based on the use of optimum percent stabilizing agent for durability and strength.

From "Comparative Performance of Structural Layer in Pavement Systems, Analysis of Test Section Data and Presentation of Design and Construction Procedures," G. M. Hammitt II, W. R. Barker, and C. L. Rone, Report RD-73-198, Vol II, Federal Aviation Administration, Washington, D. C., 1973.



NOTE: IF FROST IS A CONSIDERATION, IT IS RECOMMENDED THAT THE SUBGRADE MODULUS DURING THE THAW PERIOD BE USED IN THE DESIGN OF THIS TYPE OF PAVEMENT. THE CUMULATIVE DAMAGE CONCEPT CAN BE APPLIED IF DESIRED.

ϵ_h' AND ϵ_v' DENOTE STRAINS ADJUSTED FOR SHRINKAGE CRACKING.

Figure 3-3. Flow diagram of important events for pavement having chemically stabilized base course and unstabilized subbase course.

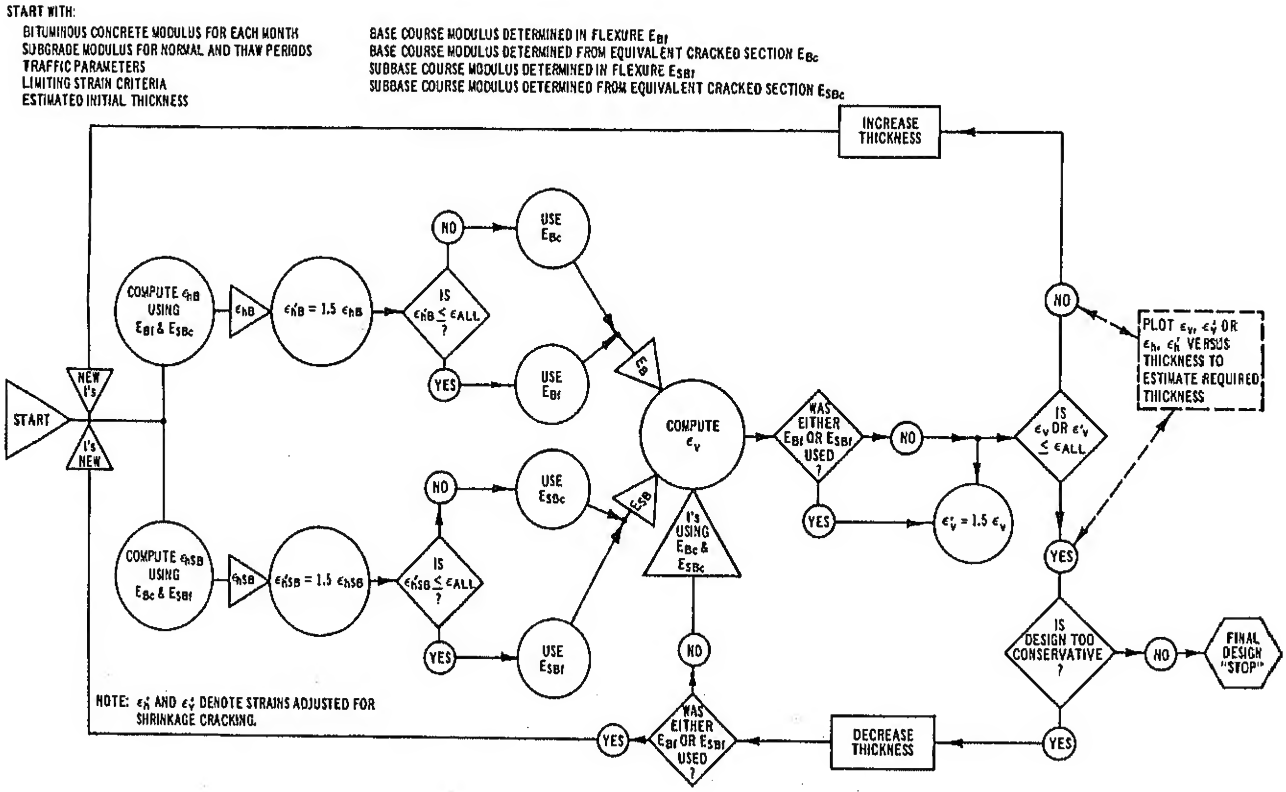


Figure 3-4. Flow diagram of important events for pavements having stabilized base and chemically stabilized subbase courses.

CHAPTER 4
CURVES FOR DETERMINING EFFECTIVE STRAIN REPETITIONS

4-1. General.

Contained in this chapter are plots (figs 4-1 through 4-30) for converting aircraft operations to effective repetitions of strain when given the type of aircraft, the effective thickness of the pavement, and the offset from the center of the runway or taxiway.

4-2. Computer Plots.

A computer program was developed by WES for producing the plots for effective strain repetitions. Should the plots not be adequate, the computer program could be used to determine the conversion factors for any design situation.

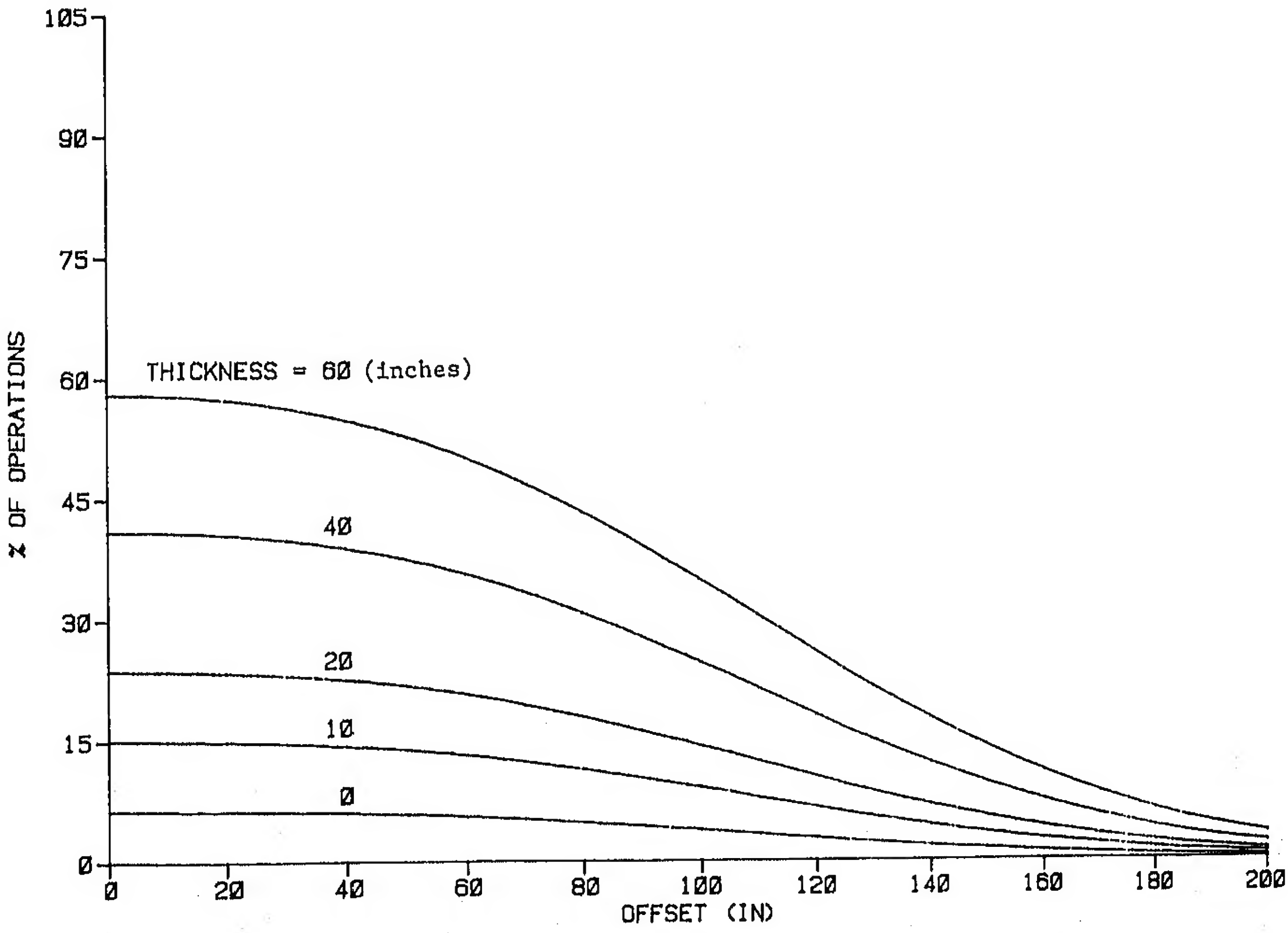


Figure 4-1. Effective repetitions of the strain for OV-1 aircraft, Army Class I airfield, type C traffic area.

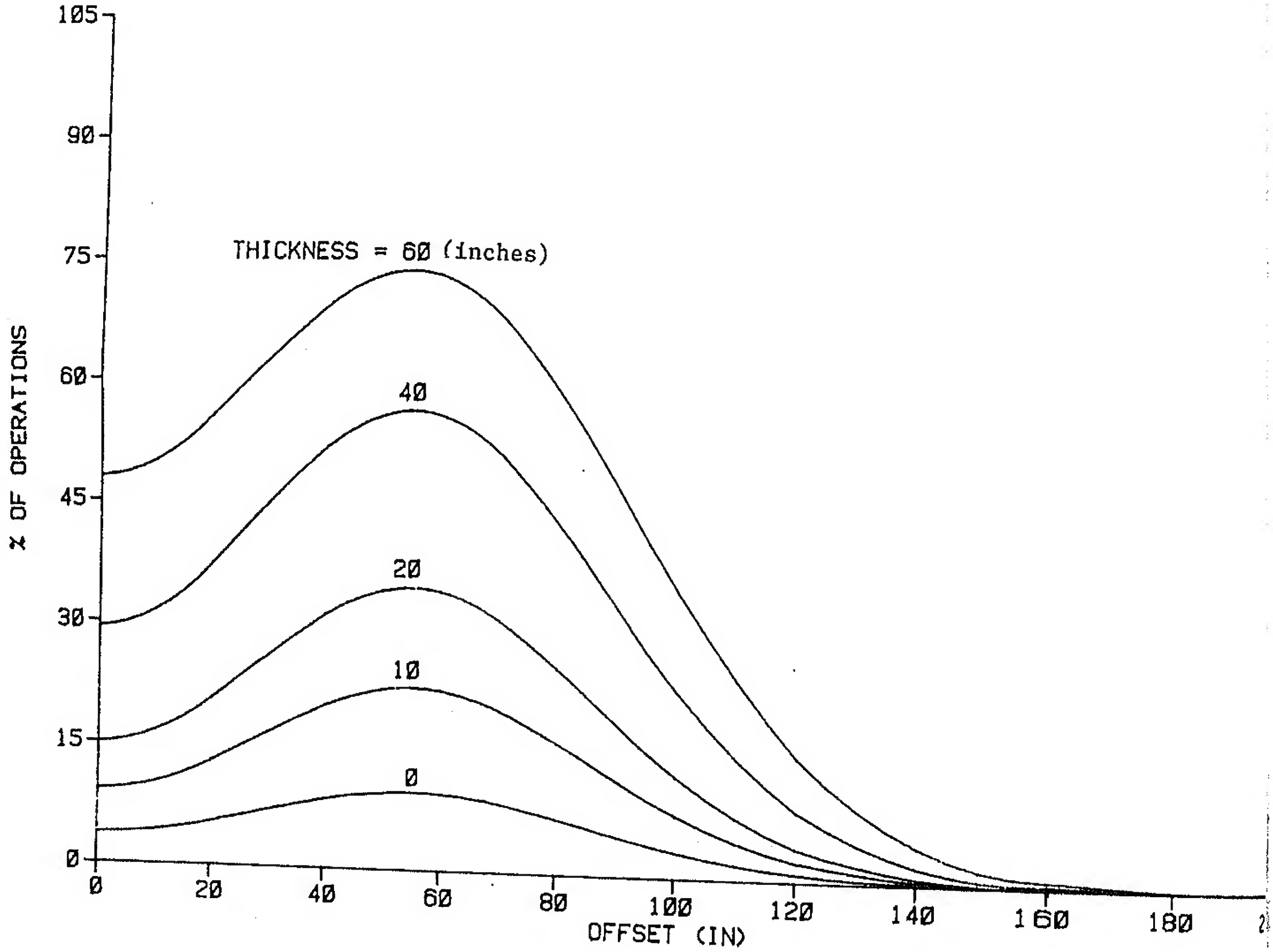


Figure 4-2. Effective repetitions of strain for OV-1 aircraft, Army Class I airfield, type B traffic area.

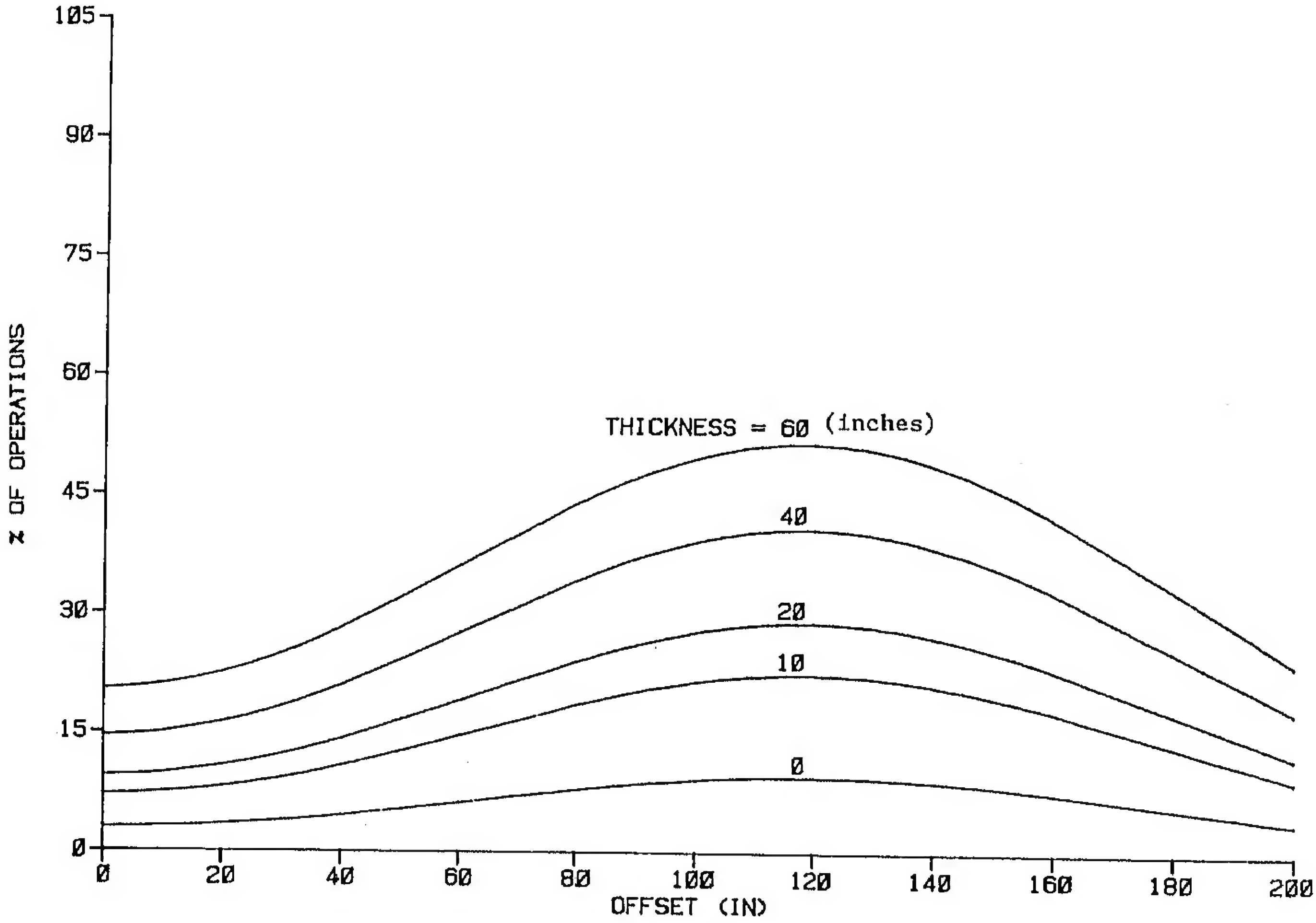


Figure 4-3. Effective repetitions of strain for CH-54 aircraft, Army Class II airfield, type C traffic area.

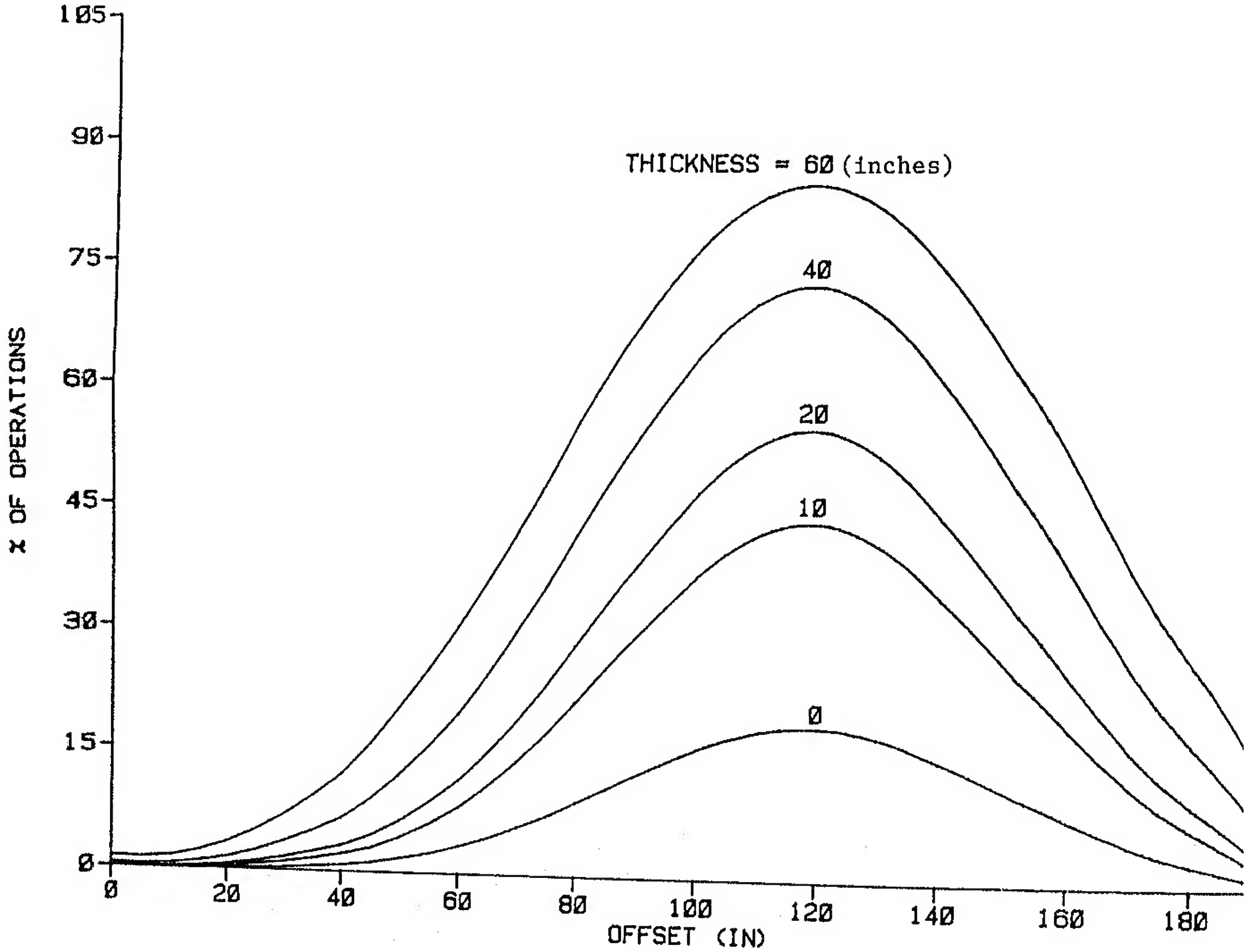


Figure 4-4. Effective repetitions of strain for CH-54 aircraft, Army Class II airfield, type B traffic area.

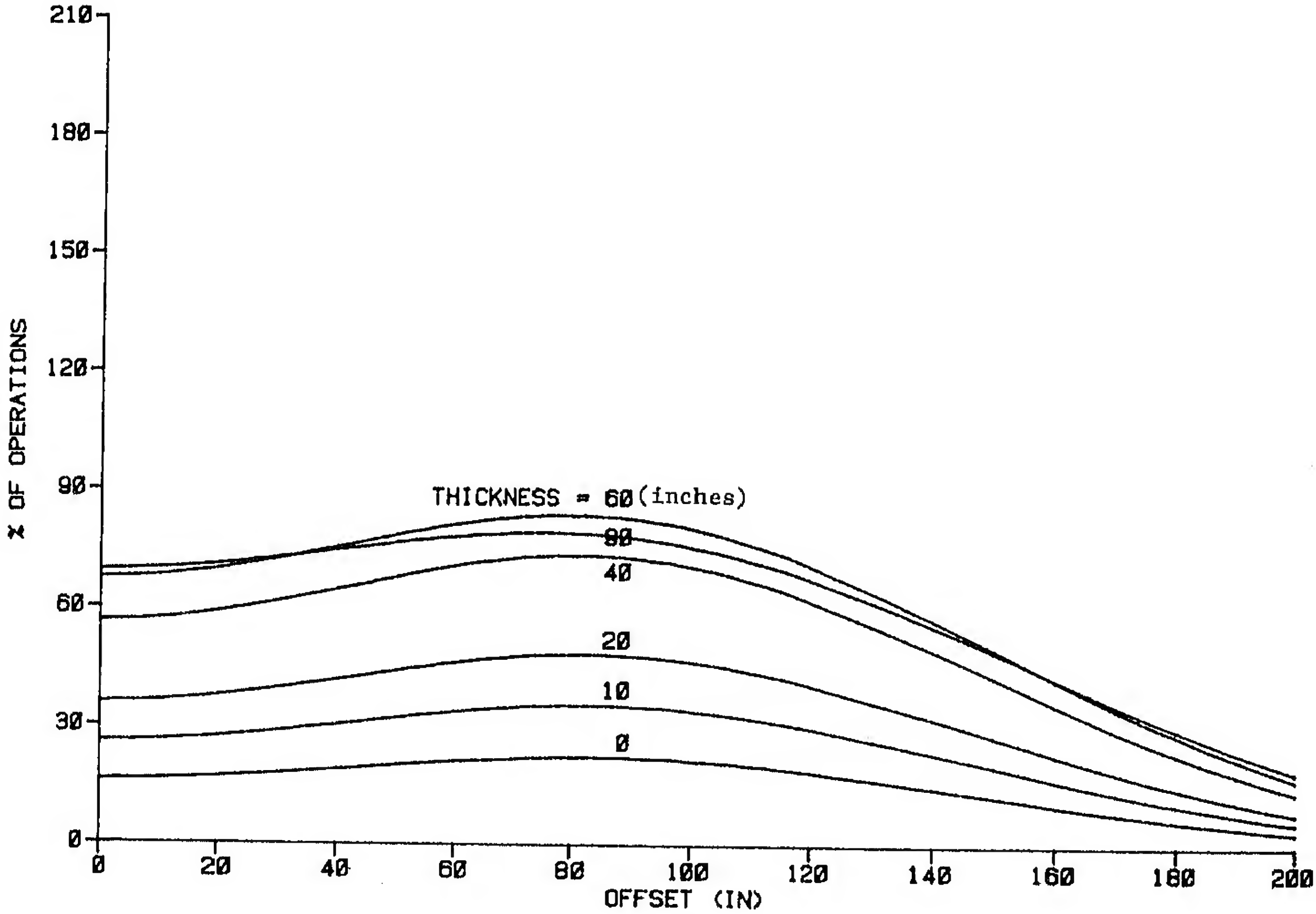


Figure 4-5. Effective repetitions of strain for C-130 aircraft, | Army Class III airfield, type C traffic area B and C traffic areas.

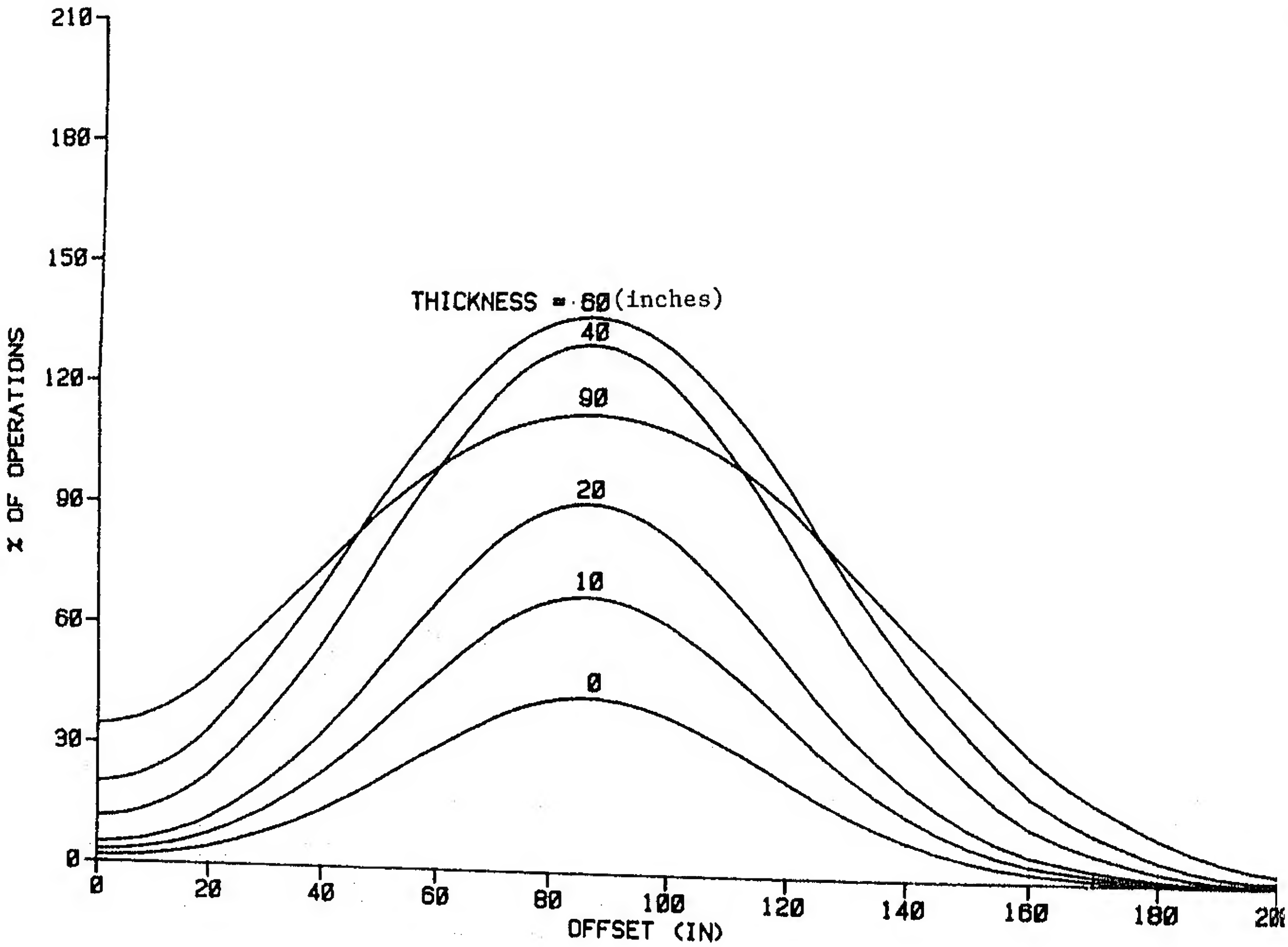


Figure 4-6. Effective repetitions of strain for C-130 aircraft, Army Class III airfield, type B traffic area, and Air Force type A traffic areas.

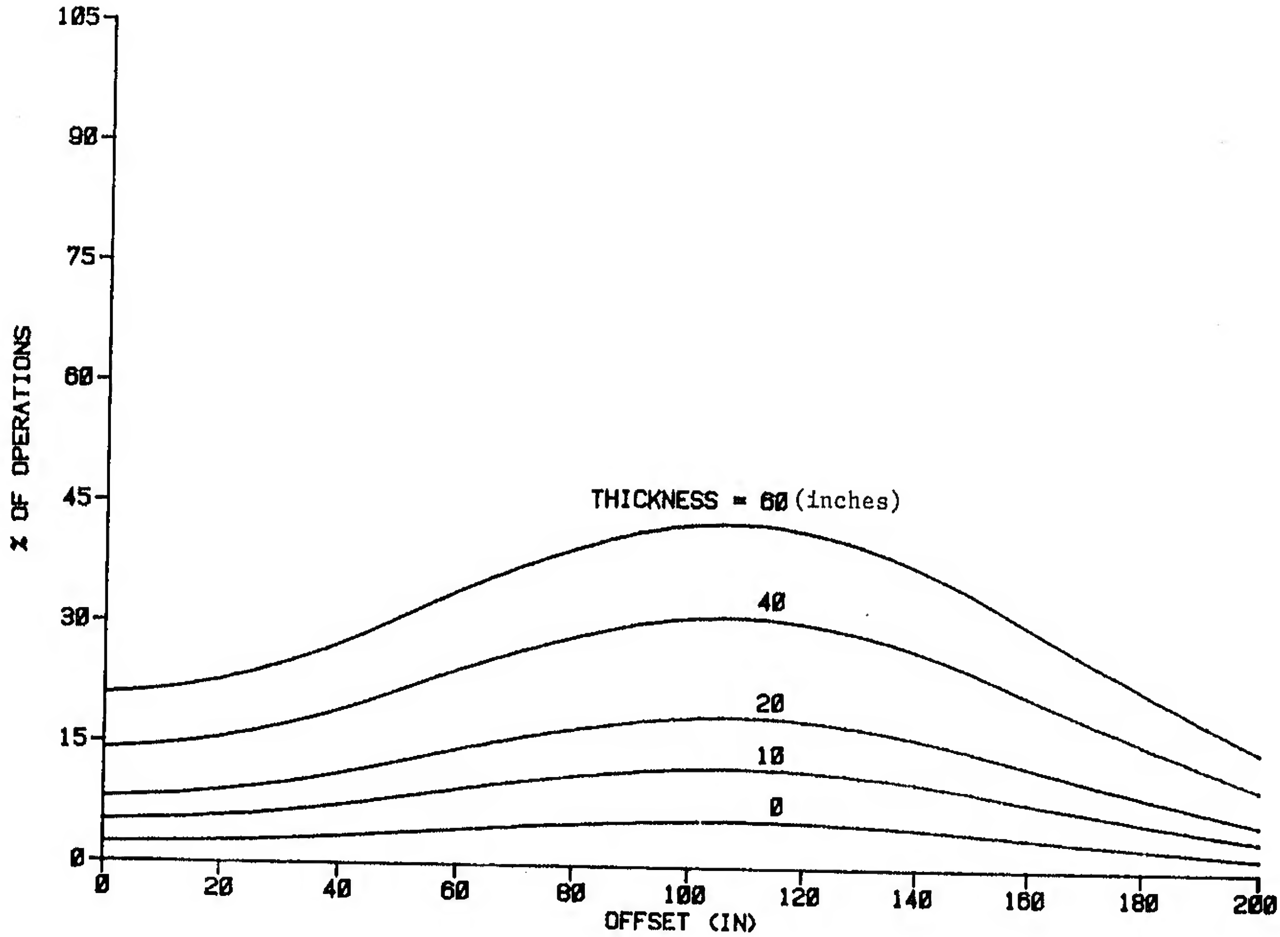


Figure 4-7. Effective repetitions of strain for F-15 and F-4 aircraft, Air Force type B and C traff

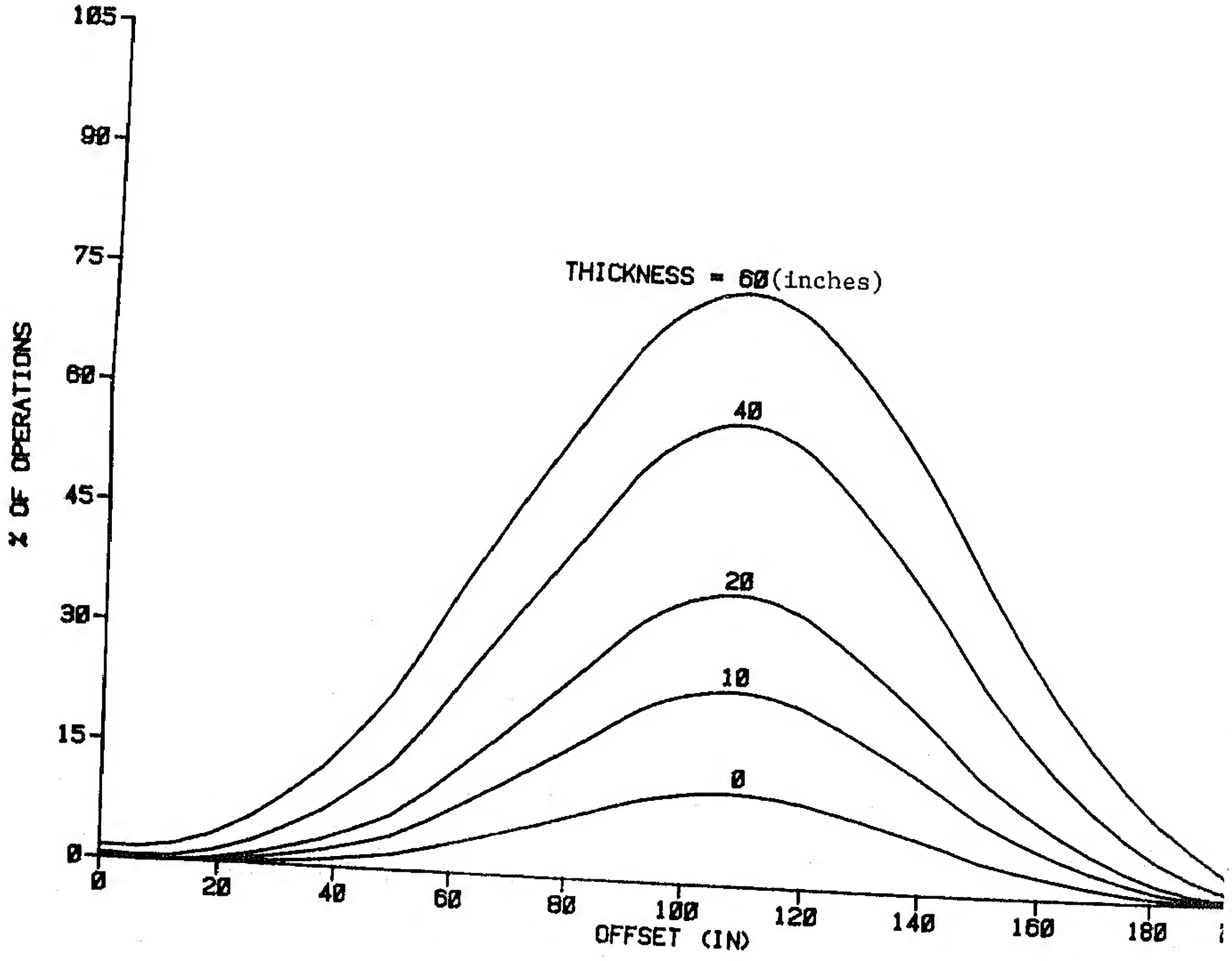


Figure 4-8. Effective repetitions of strain for F-15 and F-4 aircraft, Air Force type A traffic areas.

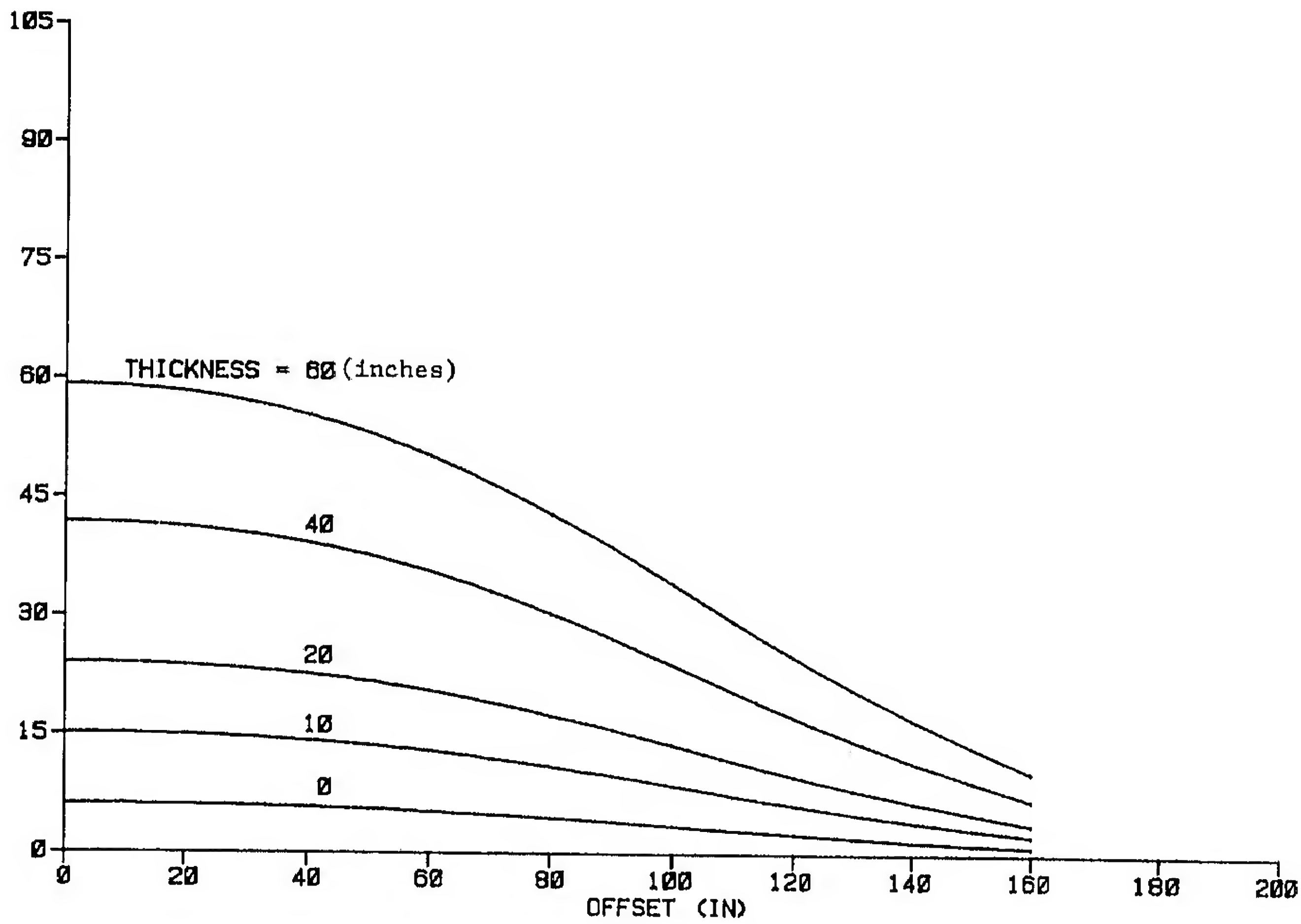


Figure 4-9. Effective repetitions of strain for F-16, F-104, and A-7 aircraft, Air Force type B and C traffic areas.

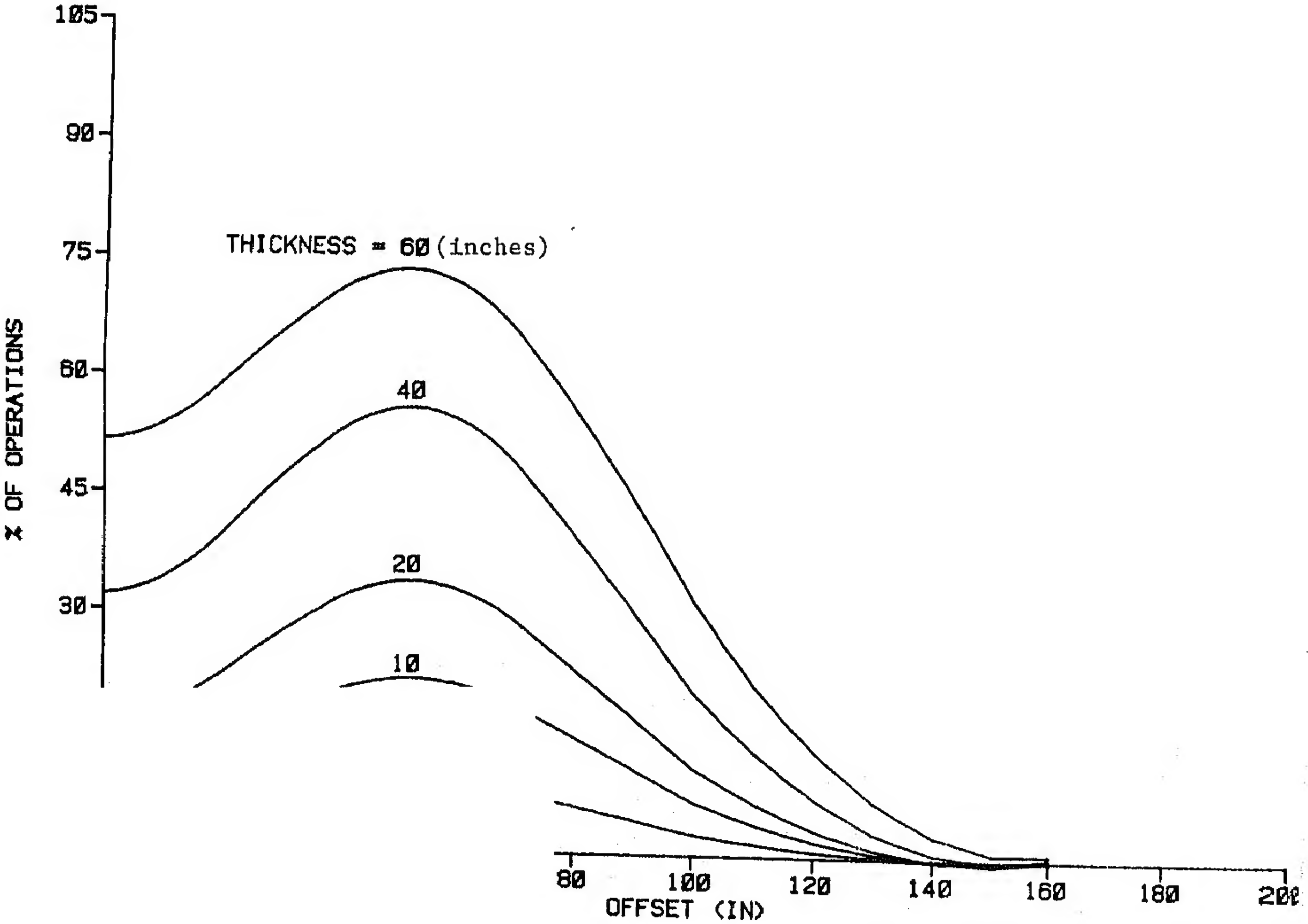


Figure 4-10. Effective repetitions of strain for F-16, F-104, and A-7 aircraft, Air Force type A traffic areas.

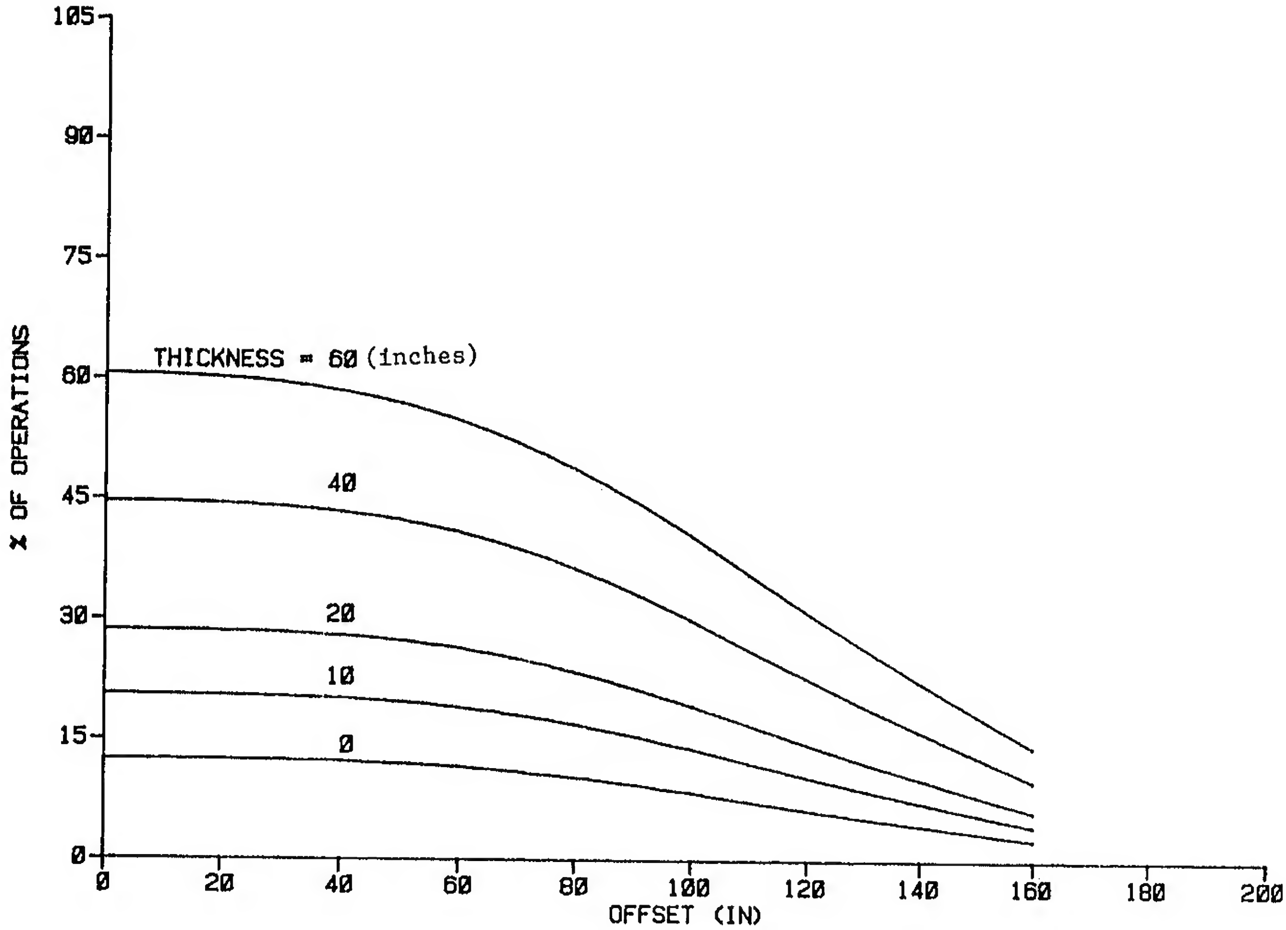


Figure 4-11. Effective repetitions of strain for F-111 aircraft, Air Force type B and C traffic areas.

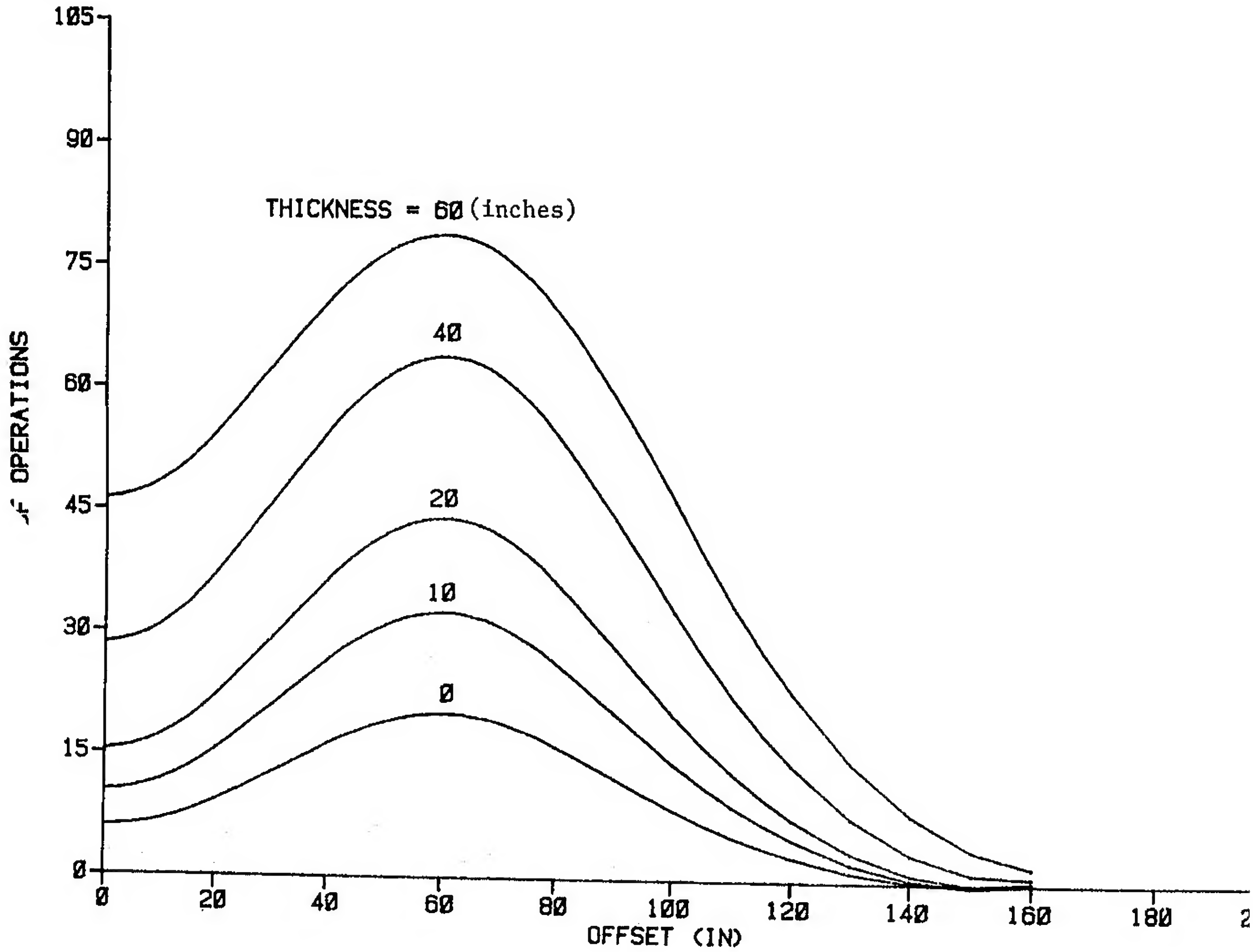


Figure 4-12. Effective repetitions of strain for F-111 aircraft, Air Force type A traffic areas.

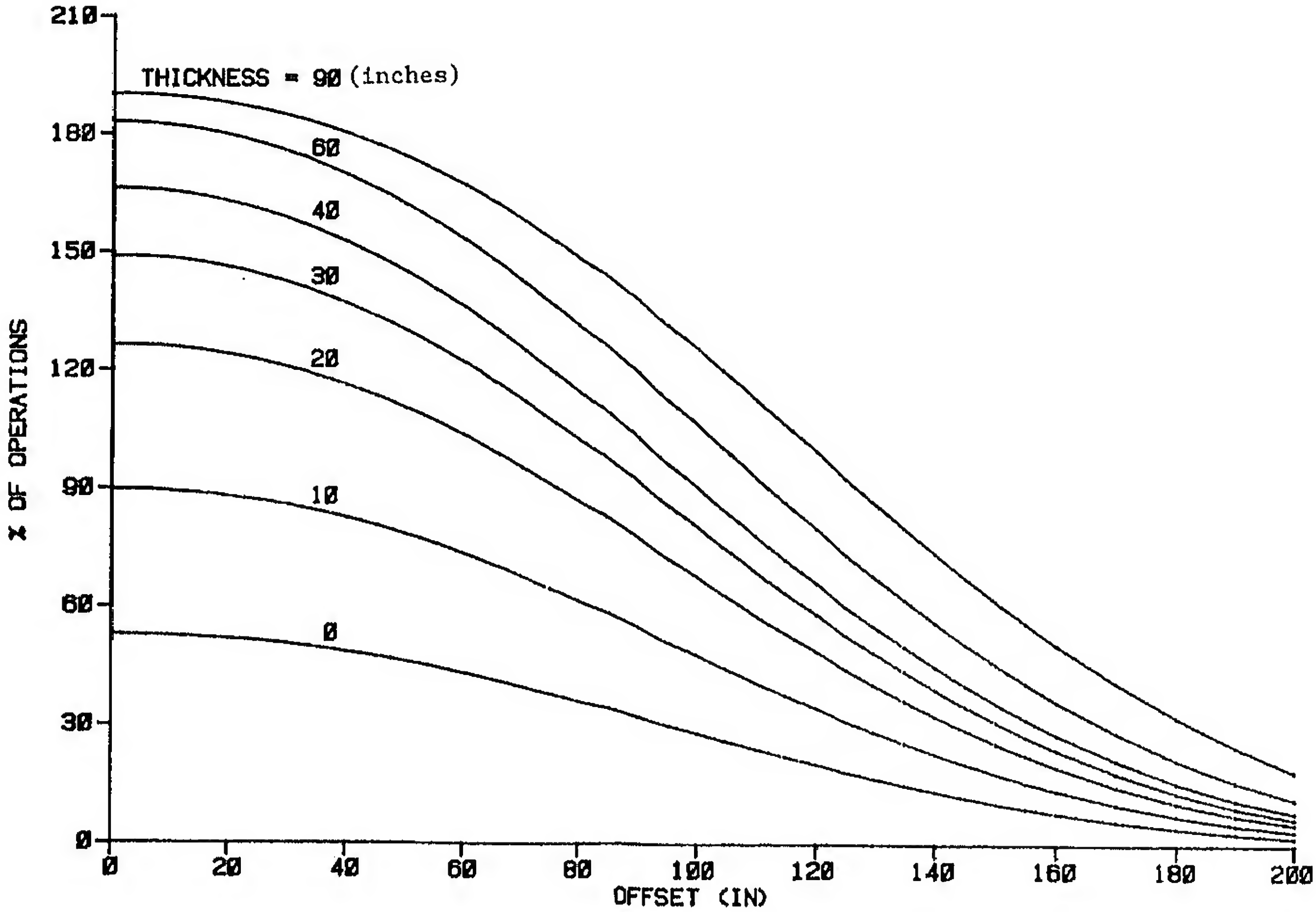


Figure 4-13. Effective repetitions of strain for B-52 aircraft, Air Force type B and C traffic areas.

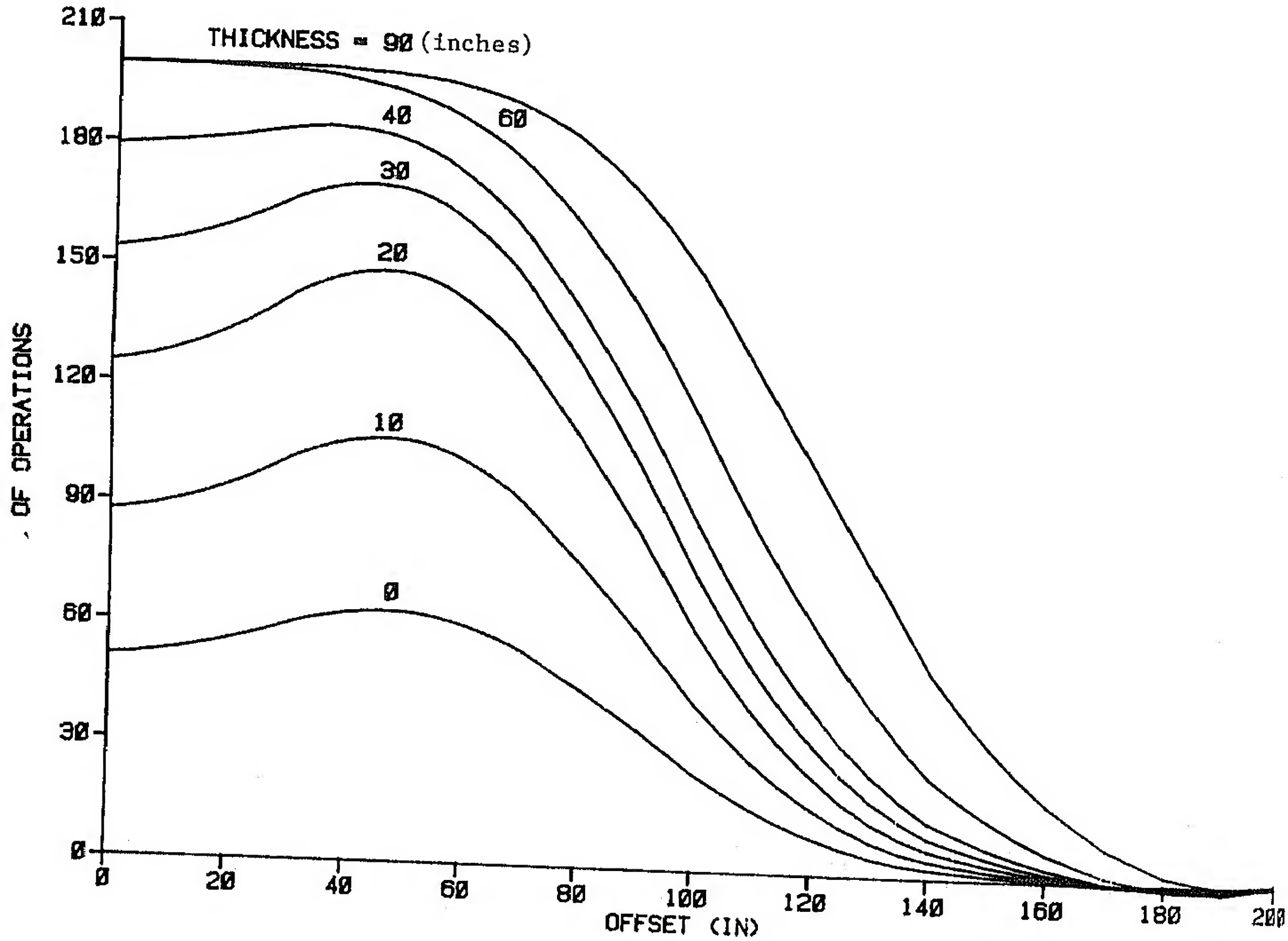


Figure 4-14. Effective repetitions of strain for B-52 aircraft, Air Force type A traffic areas.

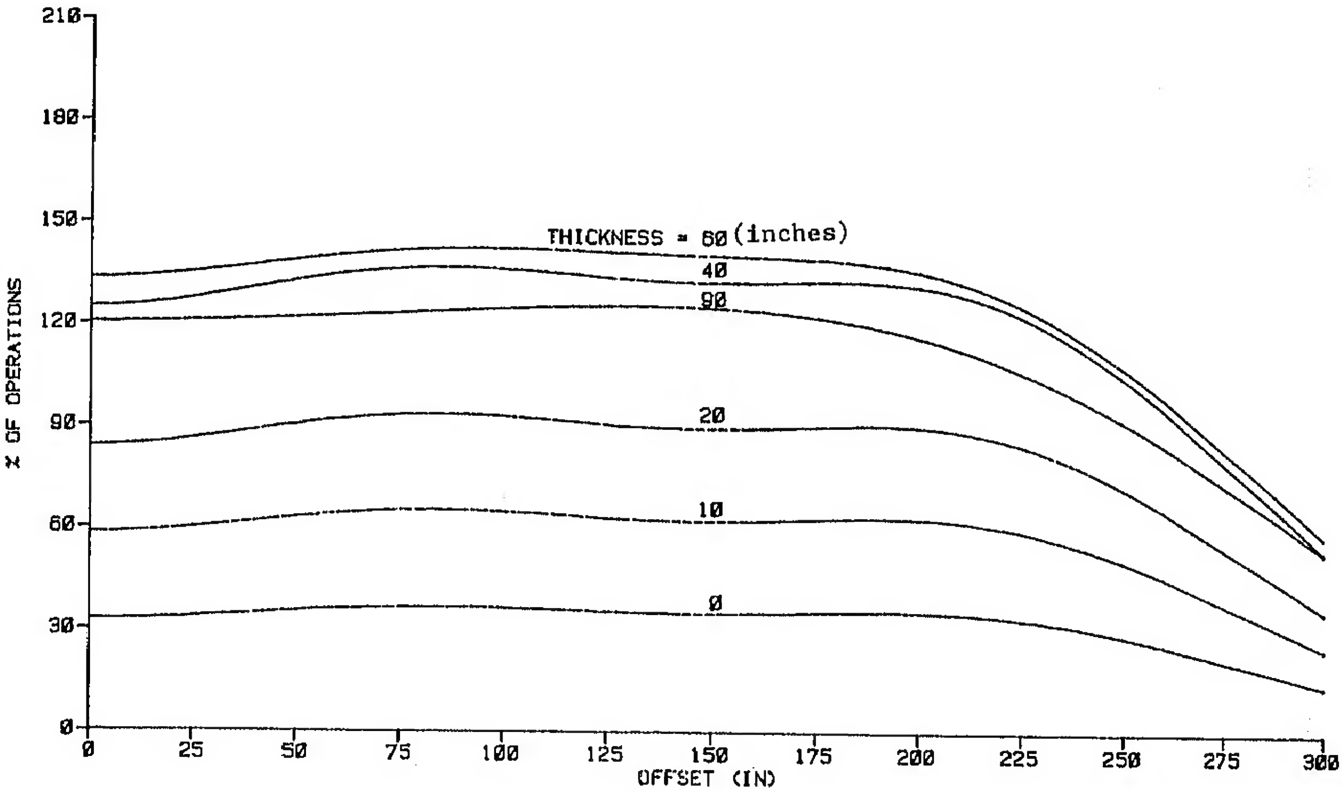


Figure 4-15. Effective repetitions of strain for B-747 aircraft, Air Force type B and C traffic areas.

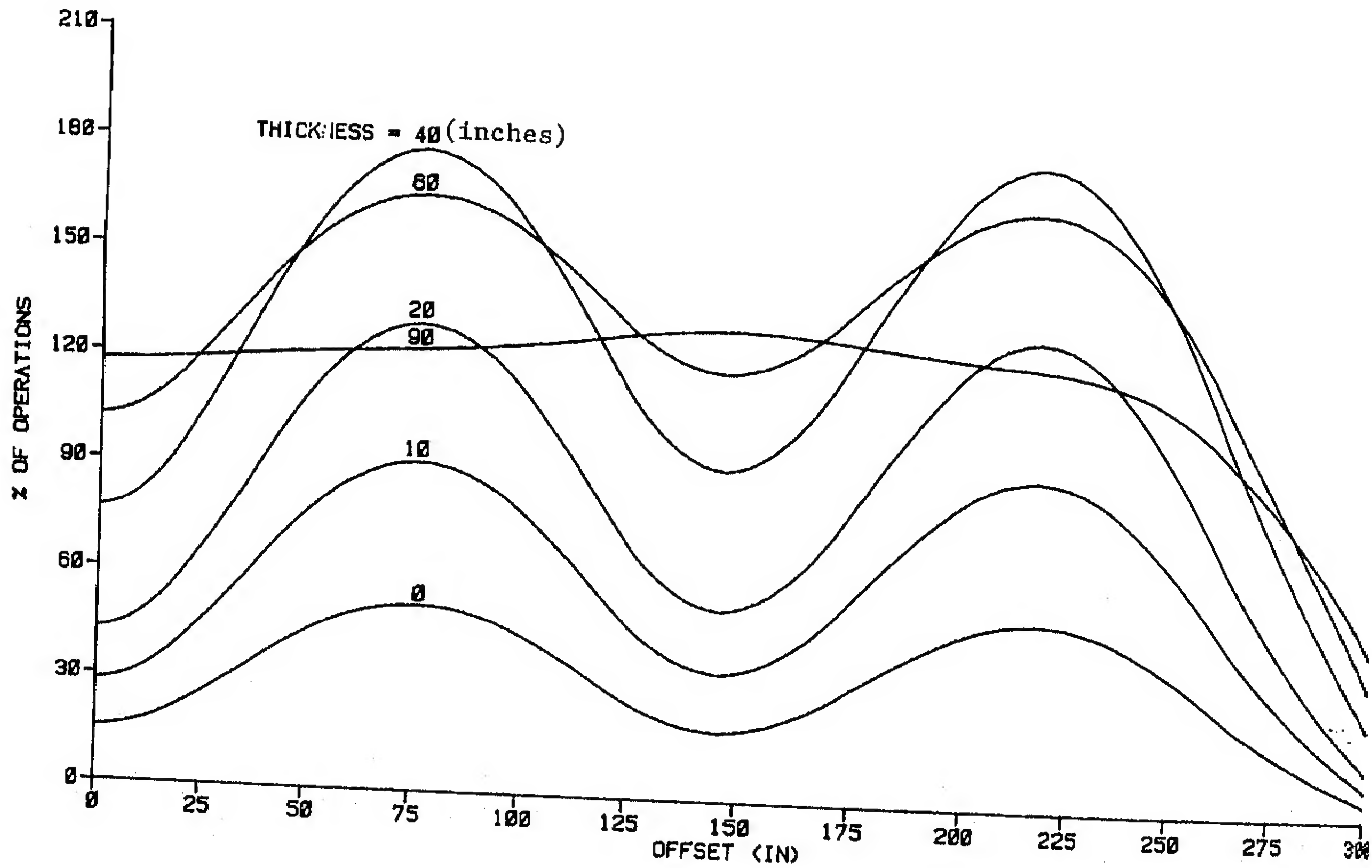


Figure 4-16. Effective repetitions of strain for B-747 aircraft, Air Force type A traffic areas.

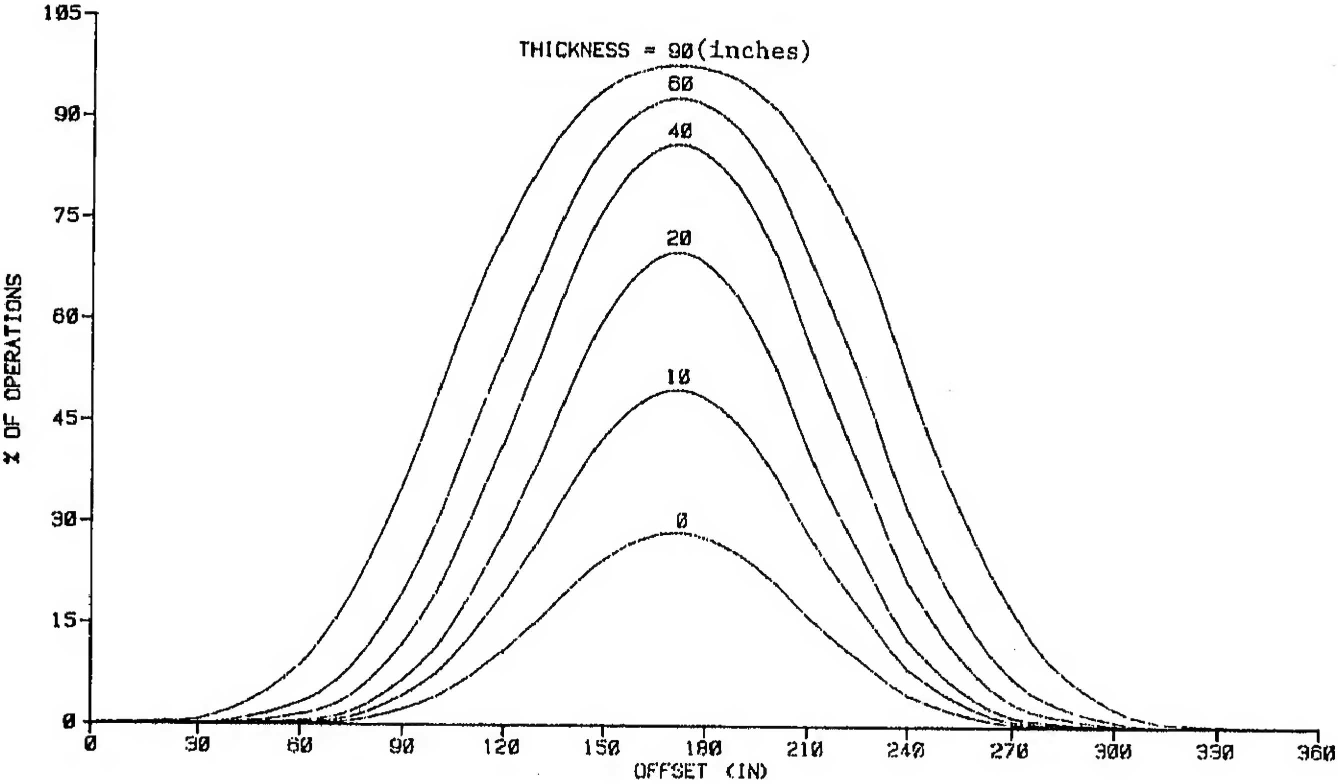


Figure 4-17. Effective repetitions of strain for C-8 and C-7 aircraft, Air Force type A traffic area.

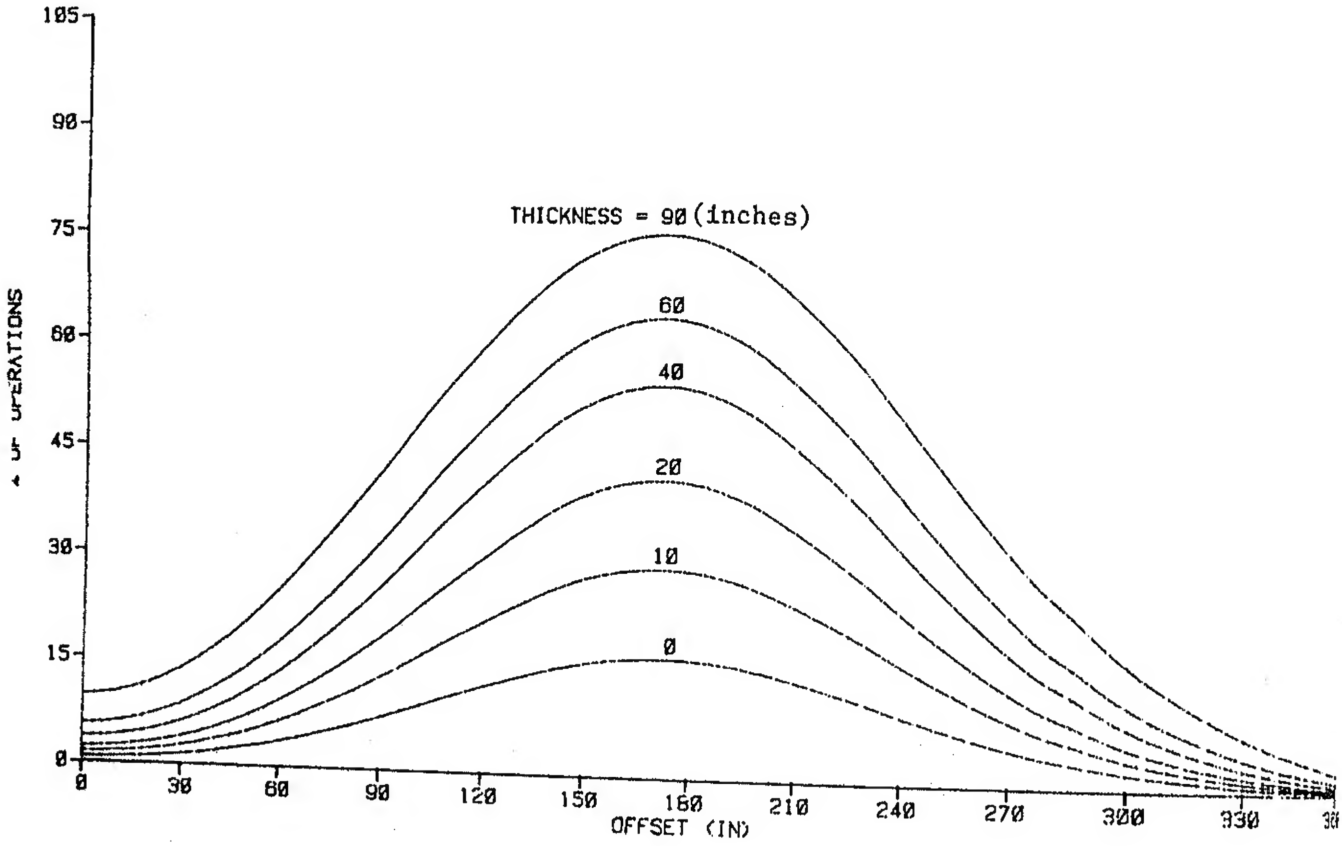


Figure 4-18. Effective repetitions of strain for C-8 and C-7 aircraft, Air Force type B and C traffic areas.

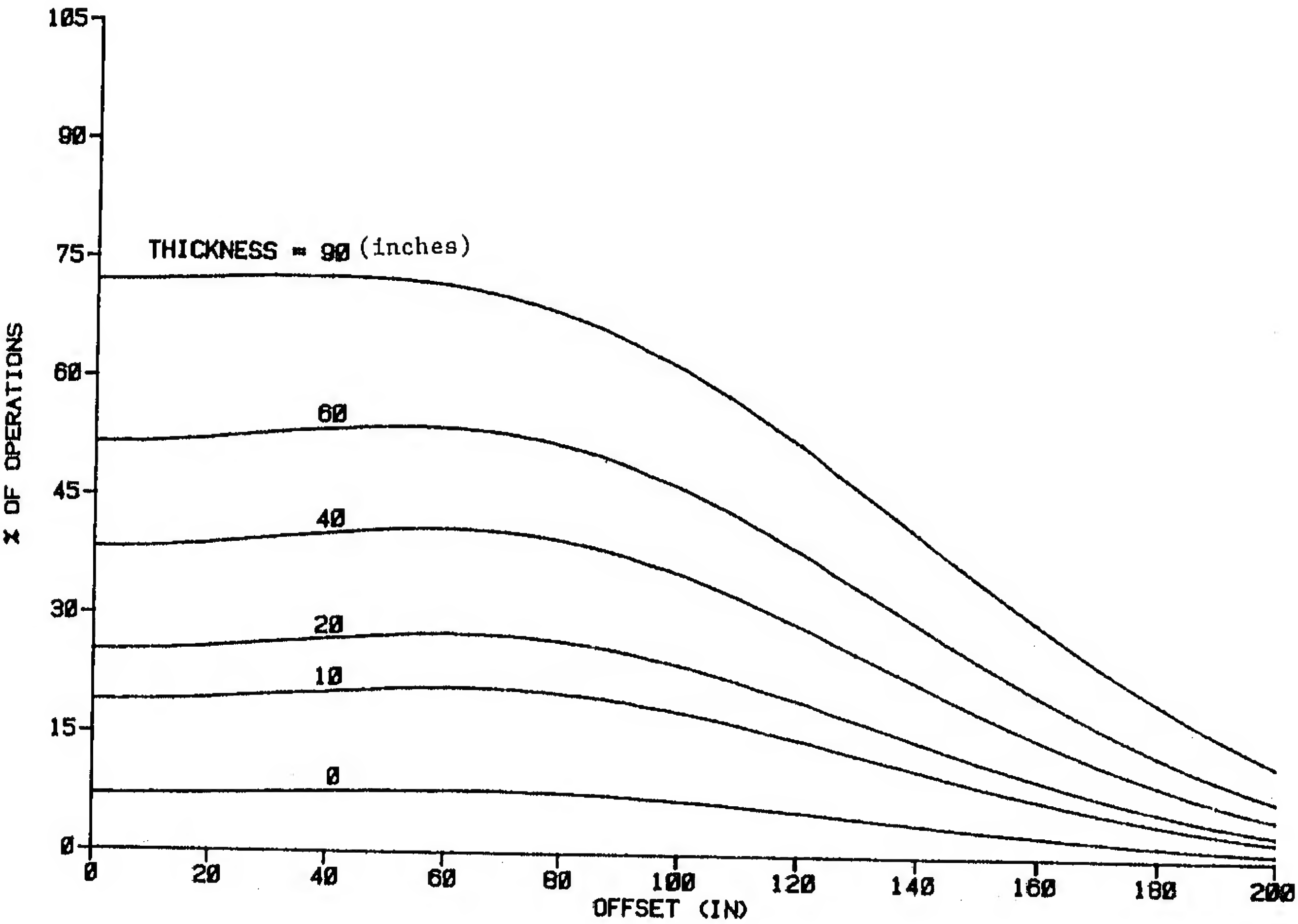


Figure 4-19. Effective repetitions of strain for C-140 aircraft, Air Force type B and C traffic areas.

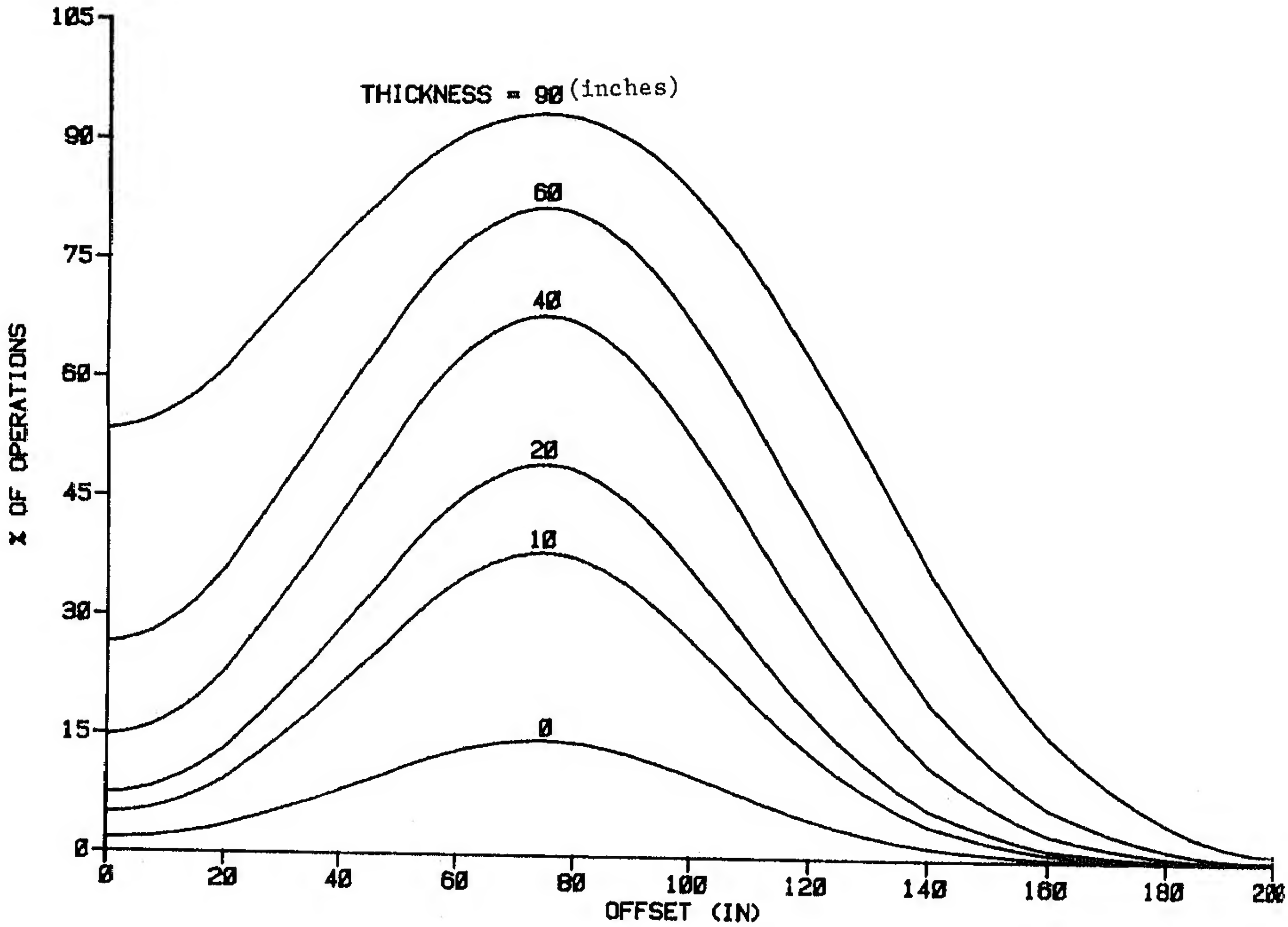


Figure 4-20. Effective repetitions of strain for C-140 aircraft, Air Force type A traffic areas.

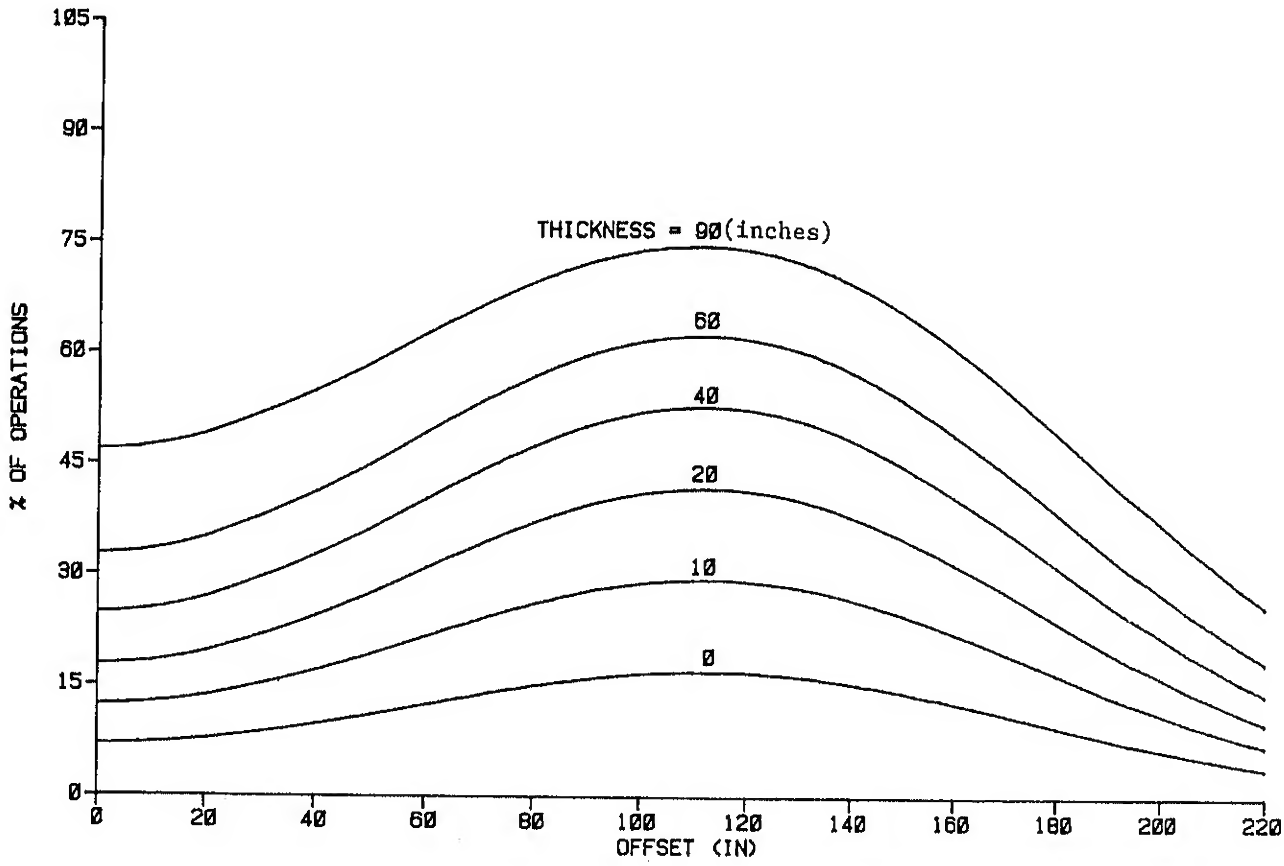


Figure 4-21. Effective repetitions of strain for B-727, B-737, and DC-9 aircraft, Air Force type B and C traffic areas.

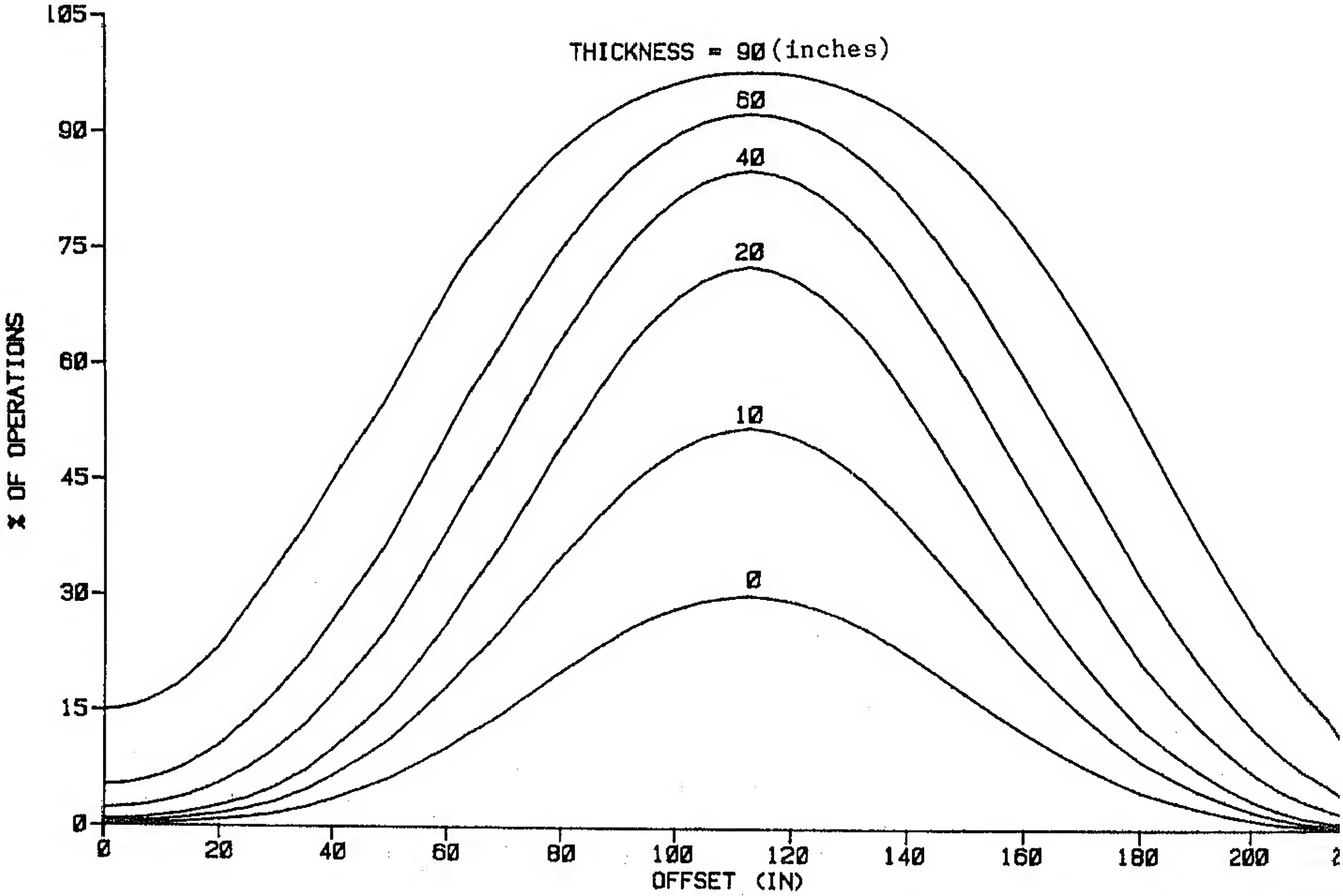


Figure 4-22. Effective repetitions of strain for B-727, B-737, and DC-9 aircraft, Air Force type A traffic areas.

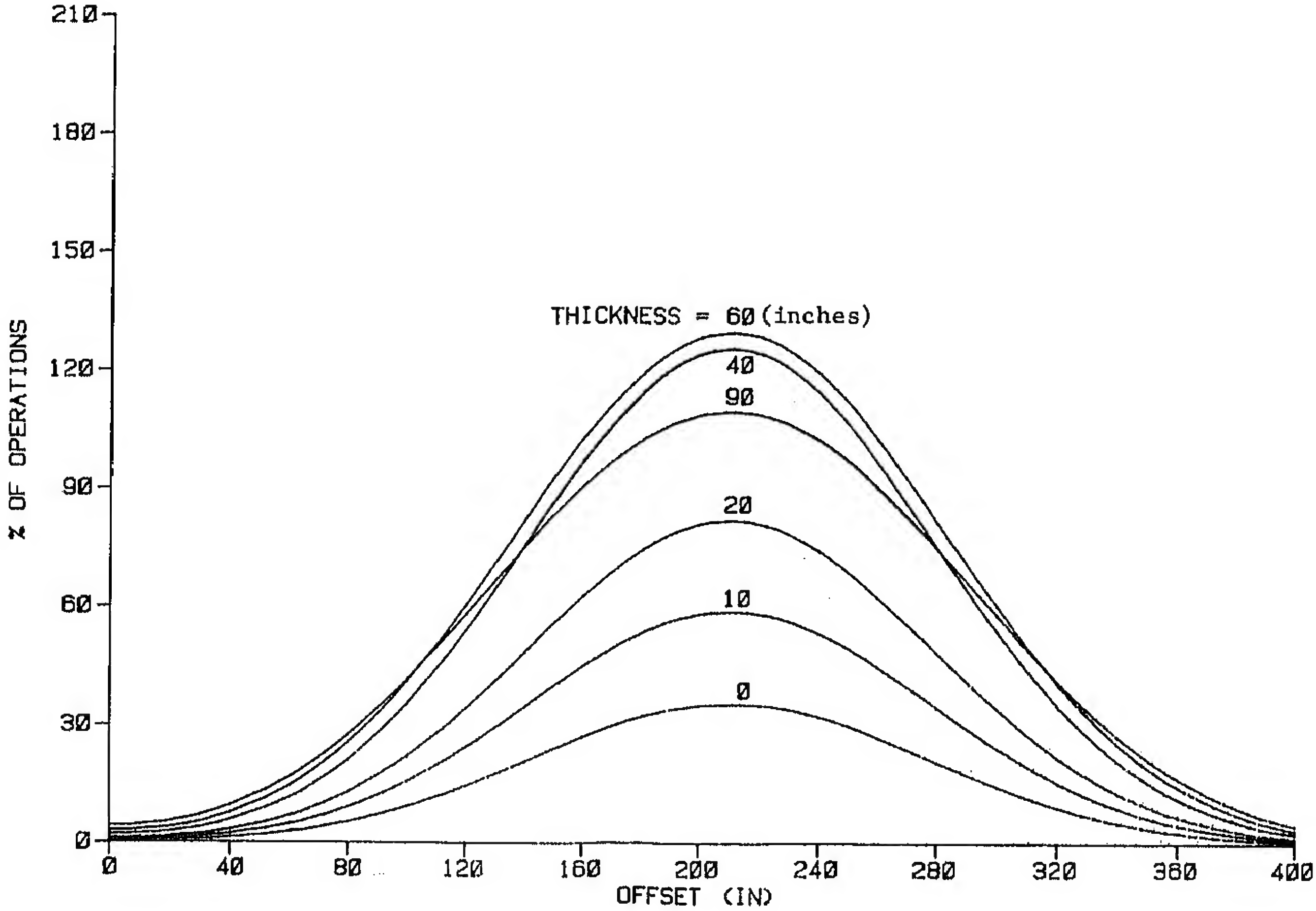


Figure 4-23. Effective repetitions of strain for DC-10, L-1011, and B-747 aircraft, Air Force type B and C traffic areas.

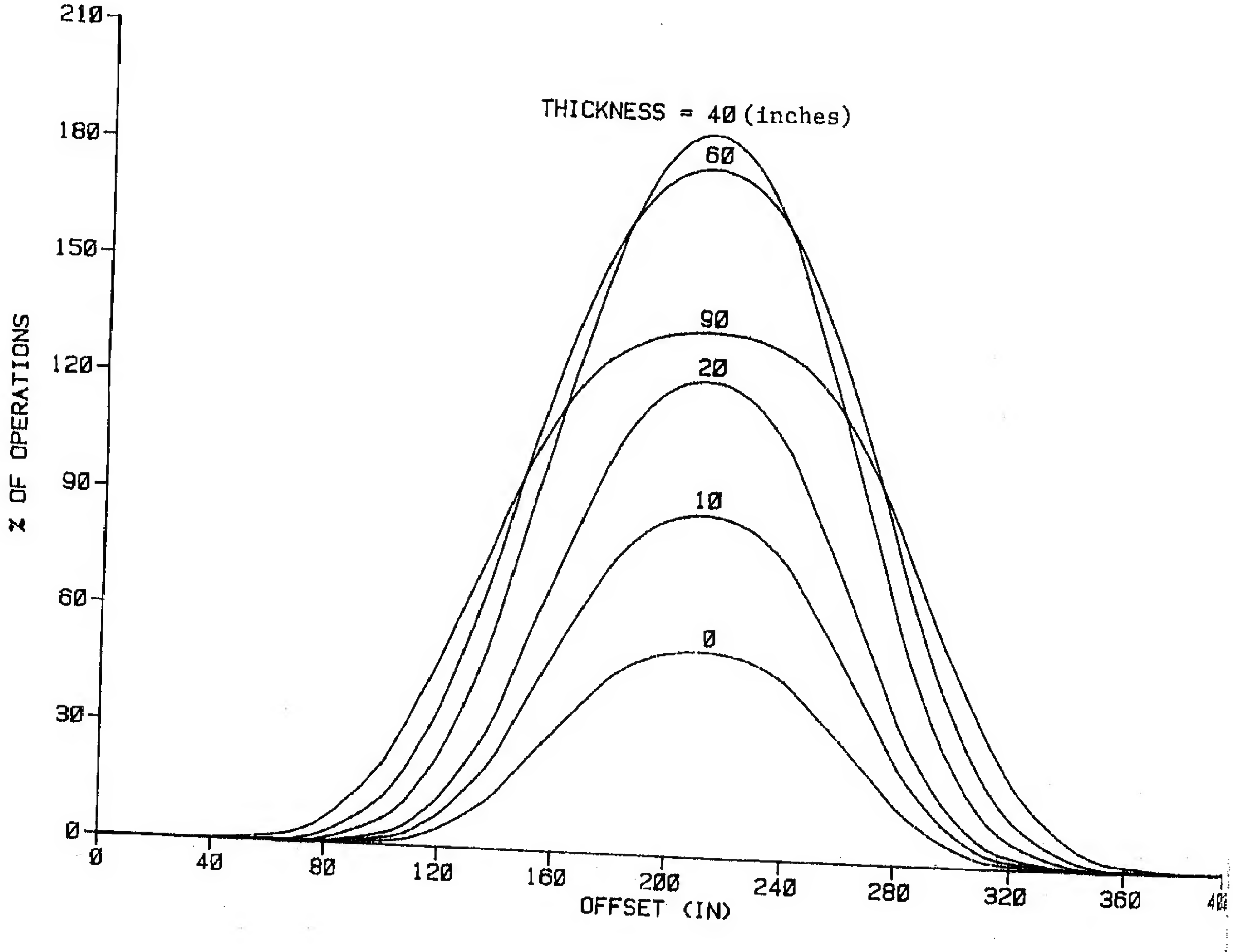


Figure 4-24. Effective repetitions of strain for DC-10, L-1011, and B-747 aircraft, Air Force type A traffic areas.

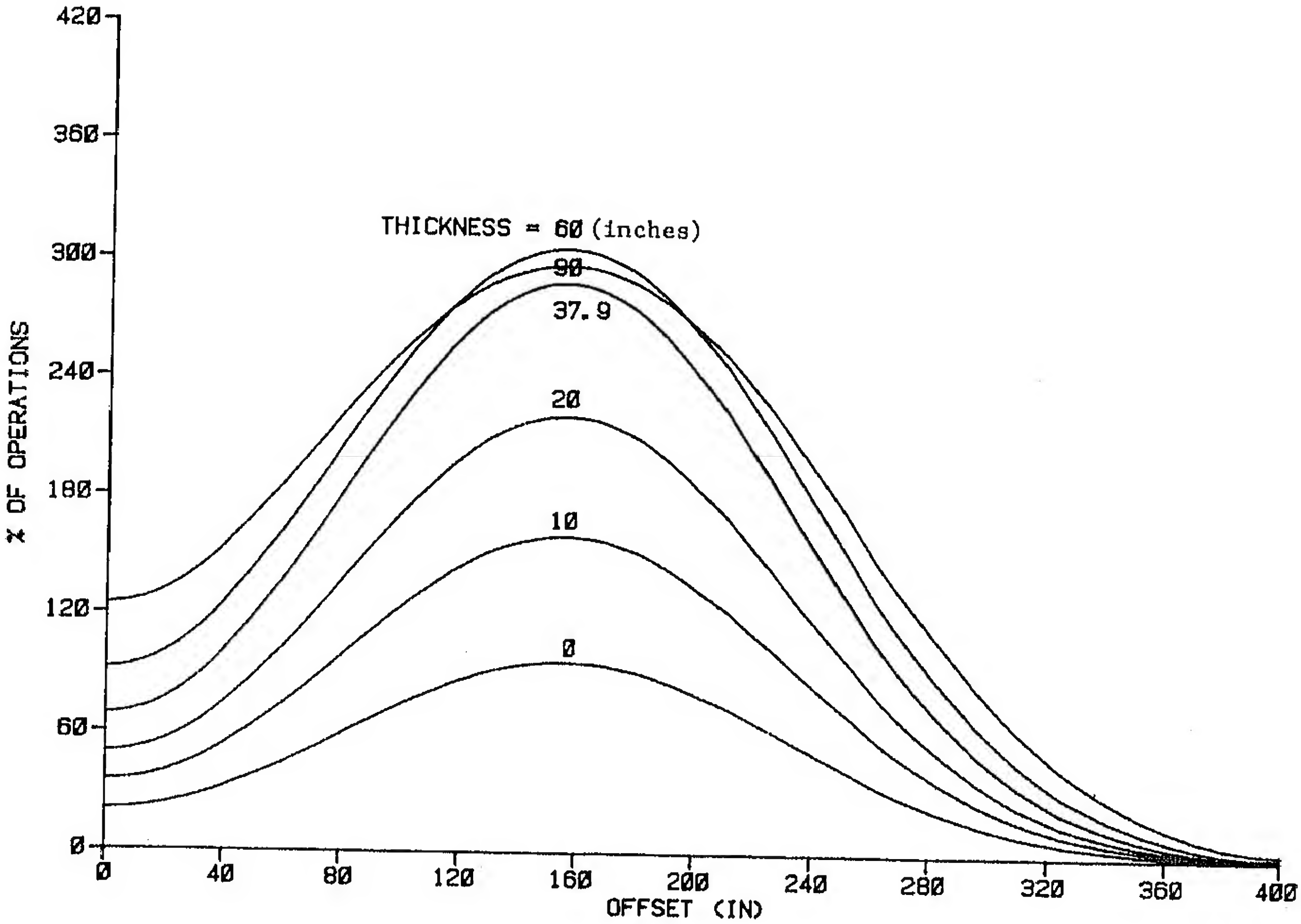


Figure 4-25. Effective repetitions of strain for C-5 aircraft, Air Force type B and C traffic areas.

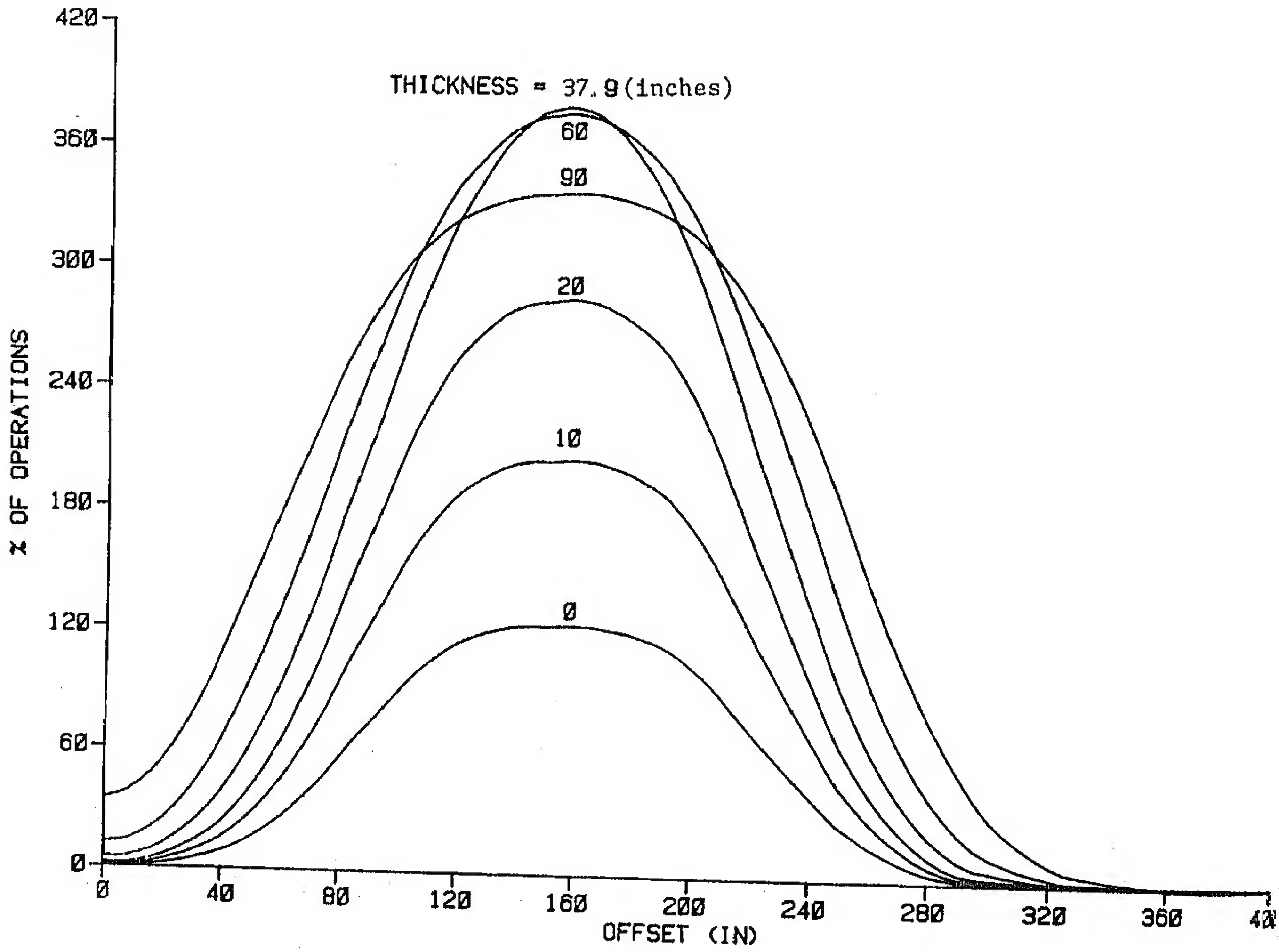


Figure 4-26. Effective repetitions of strain for C-5 aircraft, Air Force type A traffic areas.

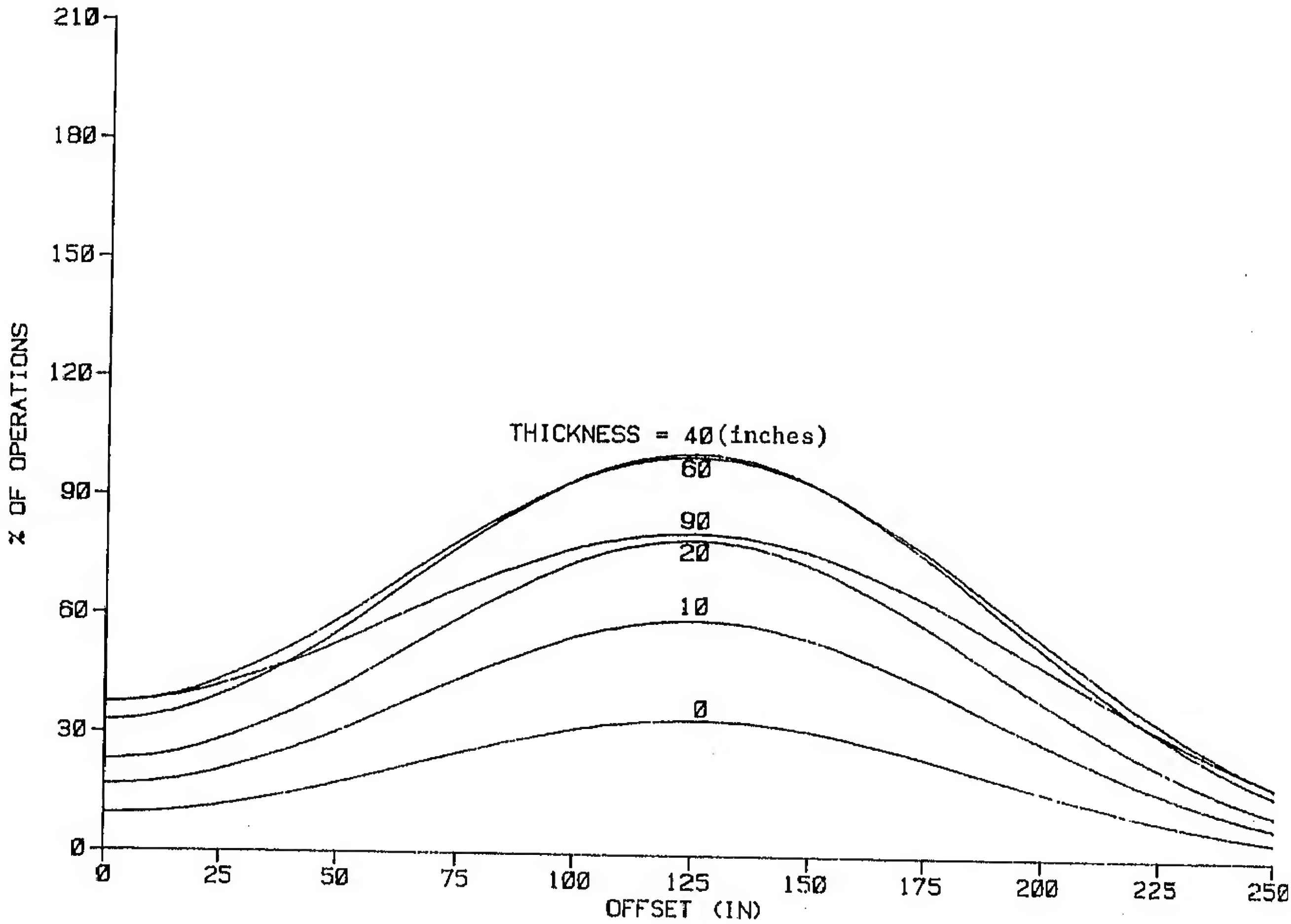


Figure 4-27. Effective repetitions of strain for E-3, B-1, B-707, KC-135, C-141, and DC-8 aircraft, Air Force

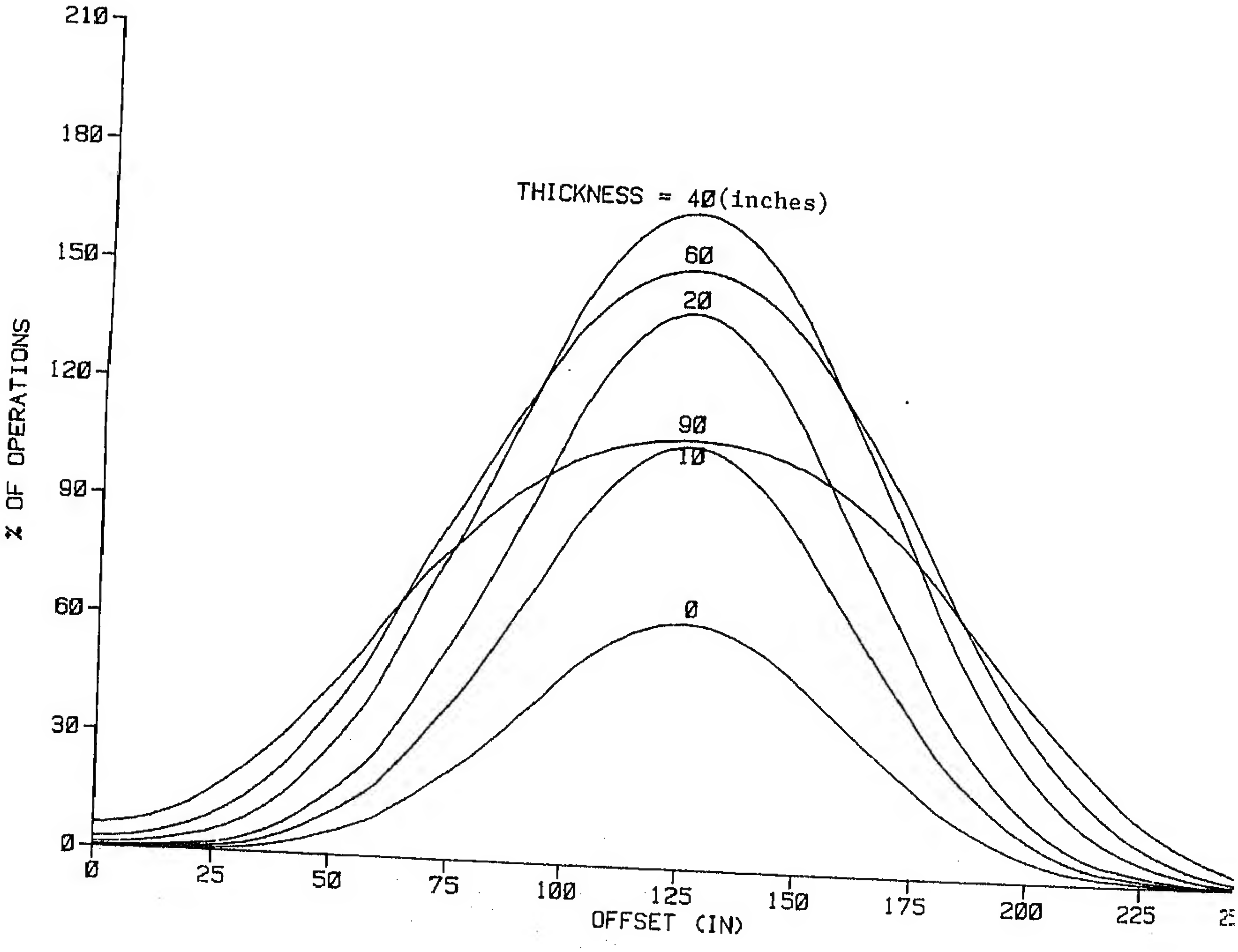


Figure 4-28. Effective repetitions of strain for E-3, B-1, B-707, KC-135, C-141, and DC-8 aircraft, Air Force type A traffic areas.

CHAPTER 5
LABORATORY PROCEDURE FOR DETERMINING THE RESILIENT MODULUS OF SUBGRADE SOILS

5-1. General.

The objective of this test procedure is to determine a modulus value for subgrade soils by means of resilient triaxial techniques. The test is similar to a standard triaxial compression test, the primary exception being that the deviator stress is applied repetitively and at several stress levels. This procedure allows testing of soil specimens in a repetitive stress state similar to that encountered by a soil in a pavement under a moving wheel load.

5-2. Definitions.

The following symbols and terms are used in the description of this procedure:

σ_1 = total axial stress

σ_3 = total radial stress; i.e., confining pressure in the triaxial test chamber

$\sigma_d = \sigma_1 - \sigma_3$ = **deviator stress; i.e., the repeated axial stress in this procedure**

ϵ_1 = **total axial stress due to** σ_d

$M_R = \sigma_d/\sigma_{R1}$ = resilient modulus

$\theta = \sigma_1 + 2\sigma_3 = \sigma_d + 3\sigma_3$ = **sum of the principal stresses in the triaxial state of stress**

σ_1/σ_3 = principal stress ratio

Load duration = time interval over which the specimen is subject to a deviator stress

Cycle duration = time interval between successive applications of a deviator stress

5-3. Specimens.

Various diameter soil specimens may be used in this test, with the specimen height at least twice the diameter. Undisturbed or laboratory molded specimens can be used. Methods for laboratory preparation of molded specimens and for back-pressure saturation of specimens are given in the following paragraphs.

5-4. Preparation of specimens.

Specimens shall have an initial height of not less than 2.1 times the initial diameter, though the minimum initial height of a specimen must be 2.25 times the diameter if the soil contains particles retained on the No. 4 sieve. The maximum particle size permitted in any specimen shall be no greater than one-sixth of the specimen diameter. Triaxial specimens 1.4, 2.8, 4, 6, 12, and 15 inches in diameter are most commonly used.

a. Cohesive soils containing negligible amounts of gravel. Specimens 1.4 inches in diameter are generally satisfactory for testing cohesive soils containing a negligible amount of gravel, while specimens of larger diameter may be advisable for undisturbed soils having marked stratification, fissures, or other discontinuities. Depending on the type of sample, specimens shall be prepared by either of the following procedures:

(1) Trimming specimens of cohesive soil. A sample that is uniform in character and sufficient in amount to provide a minimum of three specimens is required. For undisturbed soils, samples about 5 inches in diameter are preferred for triaxial tests using 1.4-inch-diameter specimens. Specimens shall be prepared in a humid room and tested as soon as possible thereafter to prevent evaporation of moisture. Extreme care shall be taken in preparing the specimens to preclude the least possible disturbance to the structure of the soil. The specimens shall be prepared as follows:

(*a*) Cut a section of suitable length from the sample. As a rule, the specimens should be cut with the long axes parallel to the long axis of the sample; any influence of stratification is commonly disregarded. However, comparative tests can be made, if necessary, to determine the effects of stratification. When a 5-inch-diameter undisturbed sample is to be used for 1.4-inch-diameter specimens, cut the sample axially into quadrants using a wire saw or other convenient cutting tool. Use three of the quadrants for specimens; seal the fourth quadrant in wax and preserve it for a possible check test.

(*b*) Carefully trim each specimen to the required diameter, using a trimming frame or similar equipment. Use one side of the trimming frame for preliminary cutting, and the other side for final trimming. Ordinarily, the specimen is trimmed by pressing the wire saw or trimming knife against the edges of the frame and cutting from top to bottom. In trimming stiff or varved clays, move the wire saw from the top and bottom toward the middle of the specimen to prevent breaking off pieces at the ends. Remove any small shells or pebbles encountered during the trimming

operations. Carefully fill voids on the surface of the specimen with remolded soil obtained from the trimming. Cut specimen to the required length (usually 3 to 3-1/2 inches for 1.4-inch-diameter specimens and 6 to 7 inches for 2.8-inch-diameter specimens) using a miter box.

(*c*) From the soil trimmings, obtain 200 grams of material for specific gravity and water content determination.

(*d*) Weigh the specimen to an accuracy of ±0.01 grams for 1.4-inch-diameter specimens and ±0.1 grams for 2.8-inch-diameter specimens.

(*e*) Measure the height and diameter of the specimen to an accuracy of ±0.01 inch. Specimen dimensions based on measurements of the trimming frame guides and miter box length are not sufficiently accurate. The average height (H_o) of the specimen should be determined from at least four measurements, while the average diameter should be determined from measurements at the top, center, and bottom of the specimen, as follows:

$$D_o = \frac{D_t + 2D_c + D_b}{4} \qquad \text{(eq 5-1)}$$

where

D_o = average diameter

D_t = diameter at top

D_c = diameter at center

D_b = diameter at bottom

(2) Compacting specimens of cohesive soil. Specimens of compacted soil may be trimmed, as described above, from samples formed in a compaction mold (a 4-inch-diameter sample is satisfactory for 1.4-inch-diameter specimens), though it is preferable to compact individual specimens in a split mold having inside dimensions equal to the dimensions of the desired specimen. The method of compacting the soil into the mold should duplicate as closely as possible the method that will be used in the field. In general, the standard impact type of compaction will not produce the same soil structure and stress-deformation characteristics as the kneading action of the field compaction equipment. Therefore, the soil should preferably be compacted into the mold (whether a specimen-size or a standard compaction mold) in at least six layers, using a pressing or kneading action of a tamper having an area in contact with the soil of less than one-sixth the area of the mold, and thoroughly scarifying the surface of each layer before placing the next. The sample shall be prepared, thoroughly mixed with sufficient water to produce the desired water content, and then stored in an airtight container for at least 16 hours. The desired density may be produced by either kneading or tamping each layer until accumulative weight of soil placed in the mold is compacted to a known volume, or adjusting the number of layers, the number of tamps per layer, and the force per tamp. For the latter method of control, special constant-force tampers (such as the Harvard miniature compactor for 1.4-inch-diameter specimens or similar compactors for 2.8-inch-diameter and larger specimens) are necessary. After each specimen compacted to finished dimensions has been removed from the mold, proceed in accordance with steps (*c*) through (*e*) of (1) above.

b. Cohesionless soils containing negligible amounts of gravel. Soils which possess little or no cohesion are difficult if not impossible to trim into a specimen. If undisturbed samples of such materials are available in sampling tubes, satisfactory specimens can usually be obtained by freezing the sample to permit cutting out suitable specimens. Samples should be drained before freezing. The frozen specimens are placed in the triaxial chamber, allowed to thaw after application of the chamber pressure, and then tested as desired. Some slight disturbance probably occurs as a result of the freezing, but the natural stratification and structure of the material are retained. In most cases, however, it is permissible to test cohesionless soils in the remolded state by forming the specimen at the desired density or at a series of densities which will permit interpolation to the desired density. Specimens prepared in this manner should generally be 2.8 inches in diameter or larger, depending on the maximum particle size. The procedure for forming the test specimen shall consist of the following steps:

(1) Oven-dry and weigh an amount of material sufficient to provide somewhat more than the desired volume of specimen.

(2) Place the forming jacket, with the membrane inside, over the specimen base of the triaxial compression device.

(3) Evacuate the air between the membrane and the inside face of the forming jacket.

(4) After mixing the dried material to avoid segregation, place the specimen, by means of a funnel or the special spoon, inside the forming jacket in equal layers. For 2.8-inch-diameter specimens, 10 layers of equal thickness are adequate. Starting with the bottom layer, compact each layer by blows with a tamping hammer, increasing the number of blows per layer linearly with the height of the layer above the bottom layer. The total number of blows required for a specimen

of a given material will depend on the density desired. Considerable experience is usually required to establish the proper procedure for compacting a material to a desired uniform density by this method. A specimen formed properly in the above-specified manner, when confined and axially loaded, will deform symmetrically with respect to its midheight, indicating that a uniform density has been obtained along the height of the specimen.

(5) As an alternate procedure, the entire specimen may be placed in a loose condition by means of a funnel or special spoon. The desired density may then be achieved by vibrating the specimen in the forming jacket to obtain a specimen of predetermined height and corresponding density. A specimen formed properly in this manner, when confined and axially loaded, will deform symmetrically with respect to its midheight.

(6) Subtract weight of unused material from original weight of the sample to obtain weight of material in the specimen.

(7) After the forming jacket is filled to the desired height, place the specimen cap on the top of the specimen, roll the ends of the membrane over the specimen cap and base, and fasten the ends with rubber bands or O-rings. Apply a low vacuum to the specimen through the base and remove the forming jacket.

(8) Measure height and diameter as specified in a(1)(*e*) above.

c. Soils containing gravel. The size of specimens containing appreciable amounts of gravel is governed by the requirements of this paragraph. If the material to be tested is in an undisturbed state, the specimens shall be prepared according to the applicable requirements of *a* and *b* above, with the size of specimen based on an estimate of the largest particle size. In testing compacted soils, the largest particle size is usually known, and the entire sample should be tested, whenever possible, without removing any of the coarser particles. However, it may be necessary to remove the particles larger than a certain size to comply with the requirements for specimen size, though such practice will result in lower measured values of the shear strength and should be avoided if possible. Oversize particles should be removed and, if comprising more than 10 percent by weight of the sample, be replaced by an equal percentage by weight of material retained on the No. 4 sieve and passing the maximum allowable sieve size. The percentage of material finer than the No. 4 sieve thus remains constant. It will generally be necessary to prepare compacted samples of material containing gravel inside a forming jacket placed on the specimen base. If the material is cohesionless, it should be oven-dried and compacted in layers inside the membrane and forming jacket using the procedure in b above as a guide. When specimens of very high density are required, the samples should be compacted preferably by vibration to avoid rupturing the membrane. The use of two membranes will provide additional insurance against possible leakage during the test as a result of membrane rupture. If the sample contains a significant amount of fine-grained material, the soil usually must possess the proper water content before it can be compacted to the desired density. Then, a special split compaction mold is used for forming the specimen. The inside dimensions of the mold are equal to the dimensions of the triaxial specimen desired. No membrane is used inside the mold, as the membrane can be readily placed over the compacted specimen after it is removed from the split mold. The specimen should be compacted to the desired density in accordance with *a*(2) above.

5-5. Q test with back-pressure saturation.

a. For the Q test with back-pressure saturation, the apparatus should be set up similar to that shown in figure 5-1. Filter strips should not be used and as little volume change as possible should be permitted during the test. Complete the steps outlined in paragraph 5-4 and the following steps:

(1) Record all identifying information for the sample project number or name, boring number, and other pertinent data on a sheet.

(2) Place one of the prepared specimens on the base.

(3) Place a rubber membrane in the membrane stretcher, turn both ends of the membrane over the ends of the stretcher, and apply a vacuum to the stretcher. Carefully lower the stretcher and membrane over the specimen. Place the specimen cap on the top of the specimen and release the vacuum on the membrane stretcher. Turn the ends of the membrane down around the base and up around the specimen cap and fasten the ends with O-rings or rubber bands. With 1.4-inch-diameter specimen of relatively insensitive soils, it is easier to roll the membrane over the specimen.

(4) Assemble the triaxial chamber and place it in position in the loading device. Connect the tube from the pressure reservoir to the base of the triaxial chamber. With valve C on the pressure reservoir closed and valves A and B open, increase the pressure inside the reservoir and allow the pressure fluid to fill the triaxial chamber. Allow a

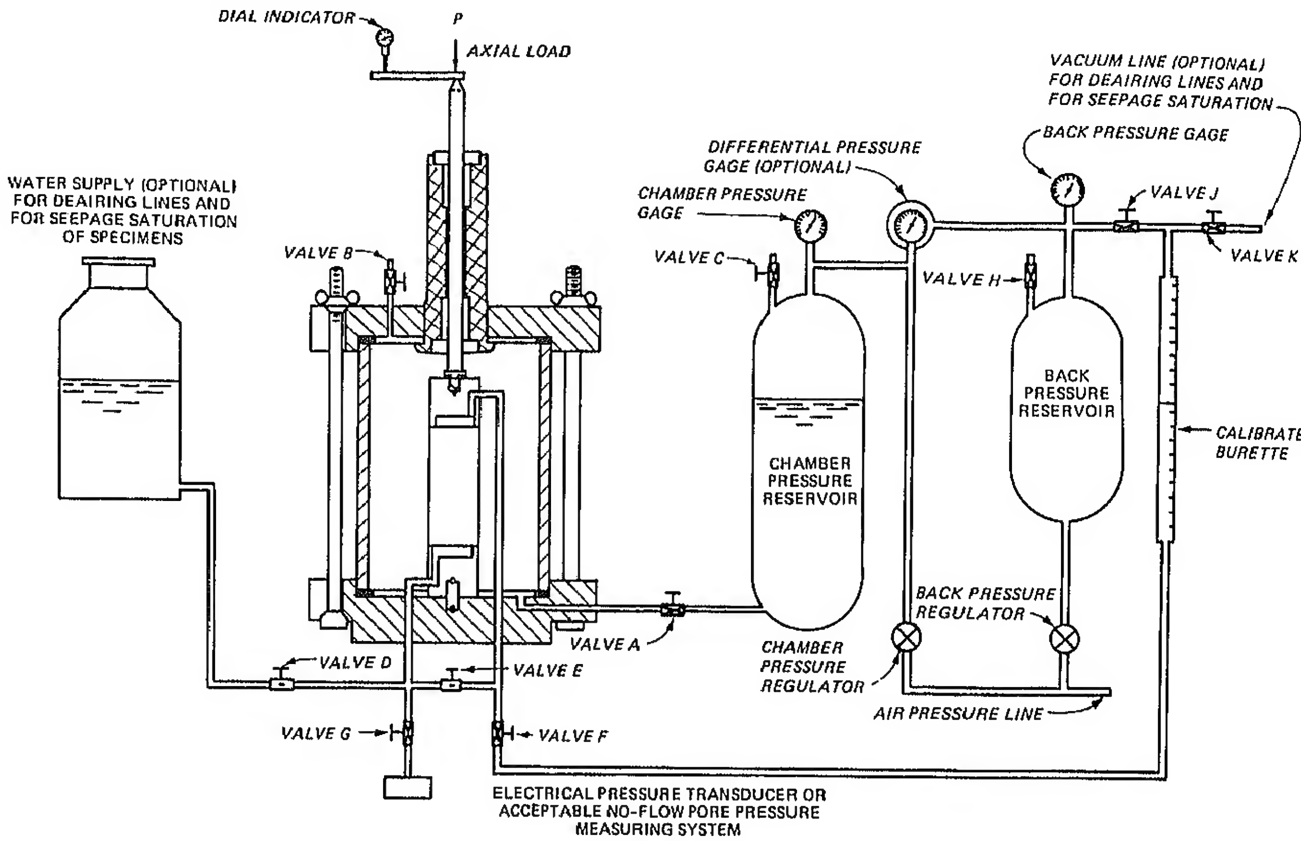


Figure 5-1. Schematic diagram of typical triaxial compression apparatus.

few drops of the pressure fluid to escape through the vent valve (valve B) to insure complete filling of the chamber with fluid. Close valve A and the vent valve.

b. Then apply 3-psi chamber pressure to the specimen with all drainage valves closed. Allow a minimum of 30 minutes for stabilization of the specimen pore water pressure, measure the change of deformation ΔH and begin back-pressure procedures as follows:

(1) Estimate the magnitude of the required back pressure by theoretical relations. Specimens should be completely saturated before any appreciable consolidation is permitted, for ease and uniformity of saturation as well as to allow volume changes during consolidation to be measured with the burette; therefore, the difference between the chamber pressure and the back pressure should not exceed 5 psi during the saturation phase. To insure that a specimen is not prestressed during the saturation phase, the back pressure must be applied in small increments, with adequate time between increments to permit equalization of pore water pressure throughout the specimen.

(2) With all valves closed, adjust the pressure regulators to a chamber pressure of about 7 psi and a back pressure of about 2 psi. Record these pressures on a data sheet. Now open valve A to apply the back pressure through the specimen cap. Immediately open valve G and read and record the pore pressure at the specimen base. When the measured pore pressure becomes essentially constant, close valves F and G and record the burette reading. (If an electrical pressure transducer is used to measure the pore pressure, valve G may be safely left open during the entire saturation procedure.)

(3) Using the technique described in *b*(2) above, increase the chamber pressure and the back pressure in increments, maintaining the back pressure at about 5 psi less than the chamber pressure. The size of each increment might be 5, 10, or even 20 psi, depending on the compressibility of the soil specimen and the magnitude of the desired consolidation pressure. Open valve G and measure the pore pressure at the base immediately upon application of each increment of back pressure and observe the pore pressure until it becomes essentially constant. The time required for stabilization of the pore pressure may range

from a few minutes to several hours depending on the permeability of the soil. Continue adding increments of chamber pressure and back pressure until, under any increment, the pore pressure reading equals the applied back pressure immediately upon opening valve G.

(4) Verify the completeness of saturation by closing valve F and increasing the chamber pressure by about 5 psi. The specimen shall not be considered completely saturated unless and the increase in pore pressure immediately equals the increase in chamber pressure.

c. After verification of saturation, and remeasurement of ΔH, close all drainage lines leading to the back pressure and pore water measurement apparatus. Holding the maximum applied back pressure constant, increase the chamber pressure until the difference between the chamber pressure and the back pressure equals the desired effective confining pressure as follows. With valves A and C closed, adjust the pressure regulator to preset the desired chamber pressure. The range of chamber pressures for the three specimens will depend on the loadings expected in the field. The maximum confining pressure should be at least equal to the maximum normal load expected in the field in order that the shear strength data need not be extrapolated for use in design analysis. Record the chamber pressure on data sheets. Now open valve A and apply the preset pressure to the chamber. Application of the chamber pressure will force the piston upward into contact with the ram of the loading device. This upward force is equal to the chamber pressure acting on the cross-sectional area of the piston minus the weight of the piston minus piston friction.

d. Then proceed as outlined below:

(1) Start the test with the piston approximately 0.1 inch above the specimen cap. This allows compensation for the effects of piston friction, exclusive of that which may later develop as a result of lateral forces. Set the load indicator to zero when the piston comes into contact with the specimen cap. In this manner the upward thrust of the chamber pressure on the piston is also eliminated from further consideration. Contact of the piston with the specimen cap is indicated by a slight movement of the load indicator. Set the strain indicator and record on the data sheet the initial dial reading at contact. Axially strain the specimen at a rate of about 1 percent per minute for plastic materials or about 0.3 percent per minute for brittle materials that achieve maximum deviator stress at about 3 to 6 percent strain; at these rates the elapsed time to reach maximum deviator stress would be about 15 to 20 minutes.

(2) Observe and record the resulting load at every 0.3 percent strain for about the first 3 percent and, thereafter, at every 1 percent, or for large strains, at every 2 percent strain; sufficient readings should be taken to completely define the shape of the stress-strain curve so frequent readings may be necessary as failure is approached. continue the test until an axial strain of 15 percent has been reached; however, when the deviator stress decreases after attaining a maximum value and is continuing to decrease at 15 percent strain, the test shall be continued to 20 percent.

(3) For brittle soils (i.e., those in which maximum deviator stress is reached at 6 percent axial strain or less), tests should be performed at rates of strain sufficient to produce times to failure as set forth in (1) above; however, when the maximum deviator stress has been clearly defined, the rate may be increased such that the remainder of the test is completed in the same length of time as that taken to reach maximum deviator stress. However, for each group of tests about 20 percent of the samples should be tested at the rates set forth in paragraphs (1) and (2) above.

(4) Upon completion of axial loading, release the chamber pressure by shutting off the air supply with the regulator and opening valve C. Open valve B and draw the pressure fluid back into the pressure reservoir by applying a low vacuum at valve C. Dismantle the triaxial chamber. Make a sketch of the specimen, showing the mode of failure.

(5) Remove the membrane from the specimen. For 1.4-inch-diameter specimens, carefully blot any excess moisture from the surface of the specimen and determine the water content of the whole specimen. For 2.8-inch-diameter or larger specimens, it is permissible to use a representative portion of the specimen for the water content determination. It is essential that the final water content be determined accurately, and weighings should be verified, preferably by a different technician.

(6) Repeat the test on the two remaining specimens at different chamber pressures, though using the same rate of strain.

5-6. Equipment.

a. Triaxial test cell.

(1) A triaxial cell suitable for use in resilience testing of soils is shown in figure 5-2. This equipment is similar to most standard cells, except that it is somewhat larger so that it can facilitate

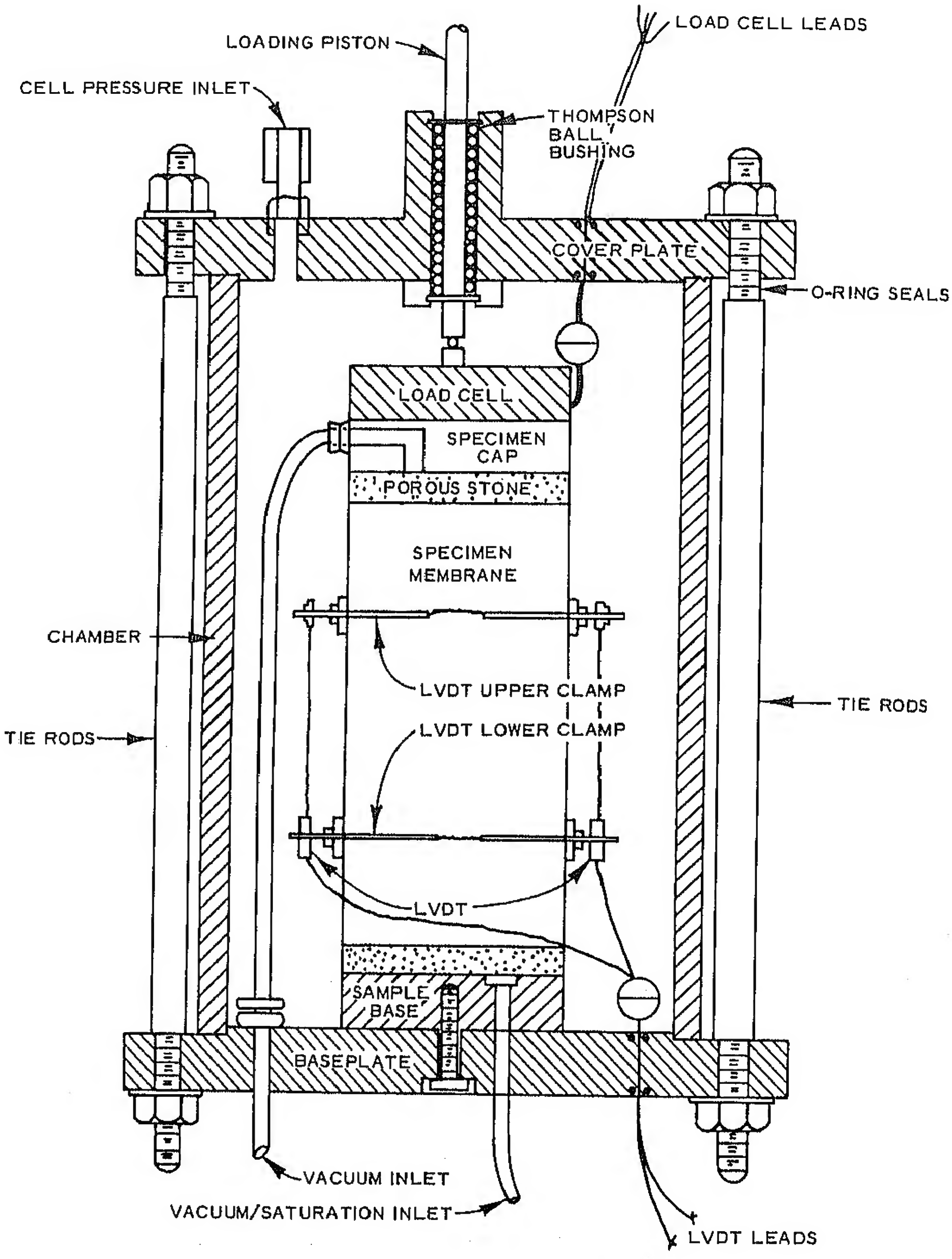


Figure 5-2. Triaxial cell.

the internally mounted load and deformation measuring equipment and the equipment has additional outlets for the electrical leads from the measuring devices. For the type of equipment shown, air or nitrogen is used as the cell fluid.

(2) The external loading source may be any device capable of providing a variable load of fixed cycle and load duration, ranging from a simple cam-and-switch control of static weights or air pistons to a closed-loop electrohydraulic system. A load duration of 0.2 seconds and a cycle duration of 3 seconds have been found to be satisfactory for most applications. A square-wave load form is recommended.

b. Deformation-measuring equipment.

(1) The deformation-measuring equipment consists of linear variable differential transducers (LVDT's) attached to the soil specimen by a pair of clamps. Two LVDT's are used for the measurement of axial deformation. The clamps and LVDT's are shown in position on a soil specimen in figure 5-1. Details of the clamps are shown in figure 5-3. Load is measured by placing a load cell between the specimen cap and the loading piston as shown in figure 5-2.

(2) Use of the type of measuring equipment described above offers several advantages:

(*a*) It is not necessary to reference deformations to the equipment, which deforms during loading.

(*b*) The effect of end-cap restrain on soil response is virtually eliminated.

(*c*) Any effects of piston friction are eliminated by measuring loads inside the triaxial cell.

(3) In addition to the measuring devices, it is also necessary to maintain suitable recording equipment. Simultaneous recording of load and deformation is desirable. The number of recording channels can be reduced by wiring the leads from the LVDT's so that only the average signal from each pair is recorded. The introduction of switching and balancing units permits use of a single-chamber recorder. However, this will not permit simultaneous recording.

c. Additional equipment. In addition to the equipment described above, the following items are also used:

— A 10- to 30-ton-capacity loading machine
— Calipers, a micrometer gage, and a steel rule

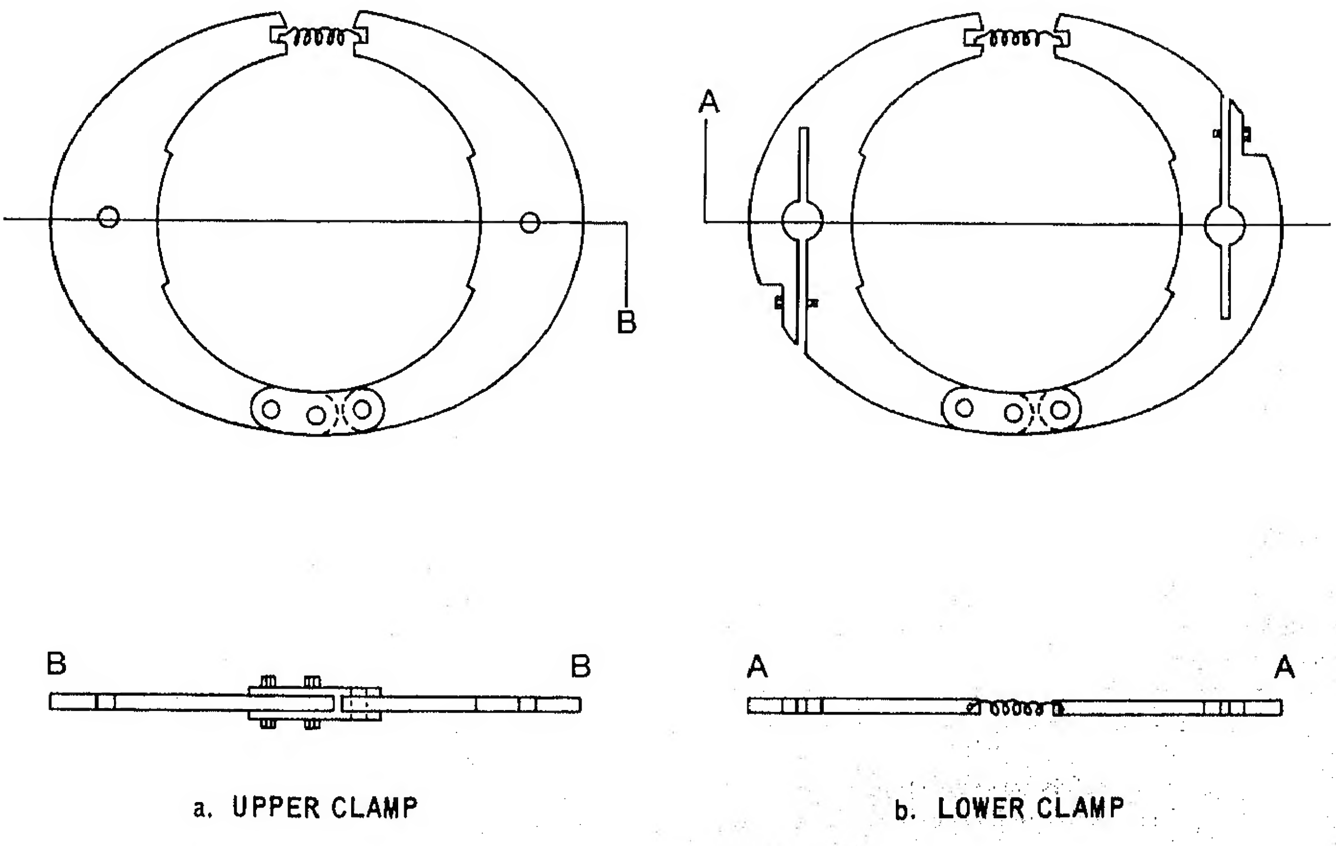


Figure 5-3. LVDT clamps.

(calibrated to 0.01 inch)

— Rubber membranes, 0.01 to 0.025 inch thick
— Rubber O-rings
— A vacuum source with a bubble chamber and regulator
— A back-pressure chamber with pressure transducers
— A membrane stretcher
— Porous stones

5-7. Preparation of specimens and placement in triaxial cell.

The following procedures shoudl be followed in preparing and placing specimens:

a. In accordance with procedures specified in paragraph 5-5, prepare the specimen and place it on the baseplate complete with porous stones, cap, and base and equipped with a rubber membrane secured with O-rings. Check for leakage. If back-pressure saturation is anticipated for cohesive soils, procedures indicated in paragraph 5-5a should be followed. For purely noncohesive soils, it will be necessary to maintain the vacuum during placement of the LVDT's. The specimen is now ready to receive the LVDT's.

b. Extend the lower LVDT clamp and slide it carefully down over the specimen to approximatly the lower third point of the specimen.

c. Repeat this step for the upper clamp, placing it at the upper third point. Insure that both clamps lie in horizontal planes.

d. Connect the LVDT's to the recording unit, and balance the recording bridges. This step will require recorder adjustments and adjustment of the LVDT stems. When a recording bridge balance has been obtained, determine (to the nearest 0.01 inch) the vertical spacing between the LVDT clamps and record this value.

e. Place the triaxial chamber in position. Set the load cell in place on the specimen.

f. Place the cover plate on the chamber. Insert the loading piston and obtain a firm connection with the load cell.

g. Tighten the tie rods firmly.

h. Slide the assembled apparatus into position under the axial loading device. Bring the loading device to a position in which it nearly contacts the loading piston.

i. If the specimen is to be back-pressure saturated, proceed in accordance with paragraph 5-5.

j. After saturation has been completed, rebalance the recorder bridge to the load cell and LVDT's.

5-8. Resilience testing of choesive soils.

a. The resilient properties of cohesive soils are only slightly affected by the magnitude of the confining pressure (σ_3). For most applications, this effect can be disregarded. When back-pressure saturation is not used, the confining pressure used should approximate the expected in situ horizontal stresses. These will generally be on the order of 1 to 5 psi. A chamber pressure of 3 psi is a reasonable value for most testing. If back-pressure saturation is used, the chamber pressure will depend on the required saturation pressure.

b. Resilient properties are highly dependent on the magnitude of the deviator stress (σ_d). It is therefore necessary to conduct the tests for a range in deviator stress values. The following procedure should be followed:

(1) If back-pressure saturation is not used, connect the chamber pressure supply line and apply the confining pressure (equal to the chamber pressure). If back-pressure saturation is used, the chamber pressure will already have been established.

(2) Rebalance the recording bridges for the LDVT's and balance the load cell recording bridge.

(3) Begin the test by applying 500 to 1,000 repetitions of a deviator stress of not more than one-half the unconfined compressive strength.

(4) Decrease the deviator load to the lowest value to be used. Apply 200 repetitions of load, recording the recovered vertical deformation at or near the last repetition.

(5) Increase the deviator load, recording deformations as in step 4. Repeat over the range of deviator stresses to be used.

(6) At the completion of the loading, reduce the chamber pressure to zero. Remove the chamber LVDT's and load cell. Use the entire specimen for the purpose of determining the moisture content.

c. The results of the resilience tests can be presented in the form of a summary table, such as table 5-1, and graphically as is shown in figure 5-4 for the resilient modulus.

5-9. Resilience testing of cohesionless soils.

a. The resilient modulus of cohesionless soils (M_R) is dependent upon the magnitude of the confining pressure (σ_3) and is nearly independent of the magnitude of the repeated axial stress. Therefore, it is necessary to test cohesionless materials over a range of confining and axial stresses. (The confining pressure is equal to the chamber pressure less the back pressure for saturated specimens.) The following procedures should be used for this type of test:

(1) Use confining pressures of 5, 10, 15, and 20 psi; at each confining pressure, test at five values of the principal stress difference corresponding to

Table 5-1. Suggested data form for recording results of resilience tests of cohesive soils

Soil Sample__________
____
____
Location__________
Sample No.__________
Specific Gravity__________

Soil Specimen Measurements

Diameter Top__________
Middle__________
Bottom__________
Average__________
Membrane Thickness__________
Net Diameter__________
Ht. Specimen + Cap + Base__________
Ht. Cap + Base__________
Initial Length L_0__________

Soil Specimen Weight

Initial Wt. of Container + Wet Soil - gms__________
Final Wt. of Container + Wet Soil - gms__________
Wt. Wet Soil Used__________

Soil Specimen Volume

Initial Area A_0 in (inch)^2__________
Volume A_0L_0 in (inch)^3__________
Wet Density - pcf__________
Water Content - %__________
% Saturation__________
Dry Density - pcf__________

Date__________

Compaction Method__________
____
Vertical Spacing Between LVDT Clamps - inch__________
Chamber Pressure - psi__________

Constants

Vertical LVDT__________

Load Cell__________

Comments__________
____
____

Load Cell Chart Reading	Deviator Load lbs	σ_d psi	Vertical LVDT Chart Reading	Vertical Deformation Inches	ϵ_{RI} in/in	$M_R = \sigma_d/\epsilon_{RI}$ psi

multiples (1, 2, 3, 4) of the cell pressure.

(2) Before beginning to record deformations, apply a series of conditioning stresses to the material to eliminate initial loading effects. The greatest amount of volume change occurs during the application of the conditioning stresses. Simulation of field conditions suggests that drainage of saturated specimens should be permitted during the application of these loads but that the test loading (beginning in step 6 below) should be conducted in an undrained state.

(3) Set the axial load generator to apply a deviator stress of 10 psi (i.e., a stress ratio equal to 3). Activate the load generator and apply 200 repetitions of this load. Stop the loading.

(4) Set the axial load generator to apply a deviator stress of 20 psi (i.e., a stress ratio equal to 5). Activate the load generator and apply 200 repetitions of this load. Stop the loading.

(5) Repeat as in step 4 above maintaining a stress ratio equal to 6 and using the following order and magnitude of confining pressures: 10, 20, 10, 5,

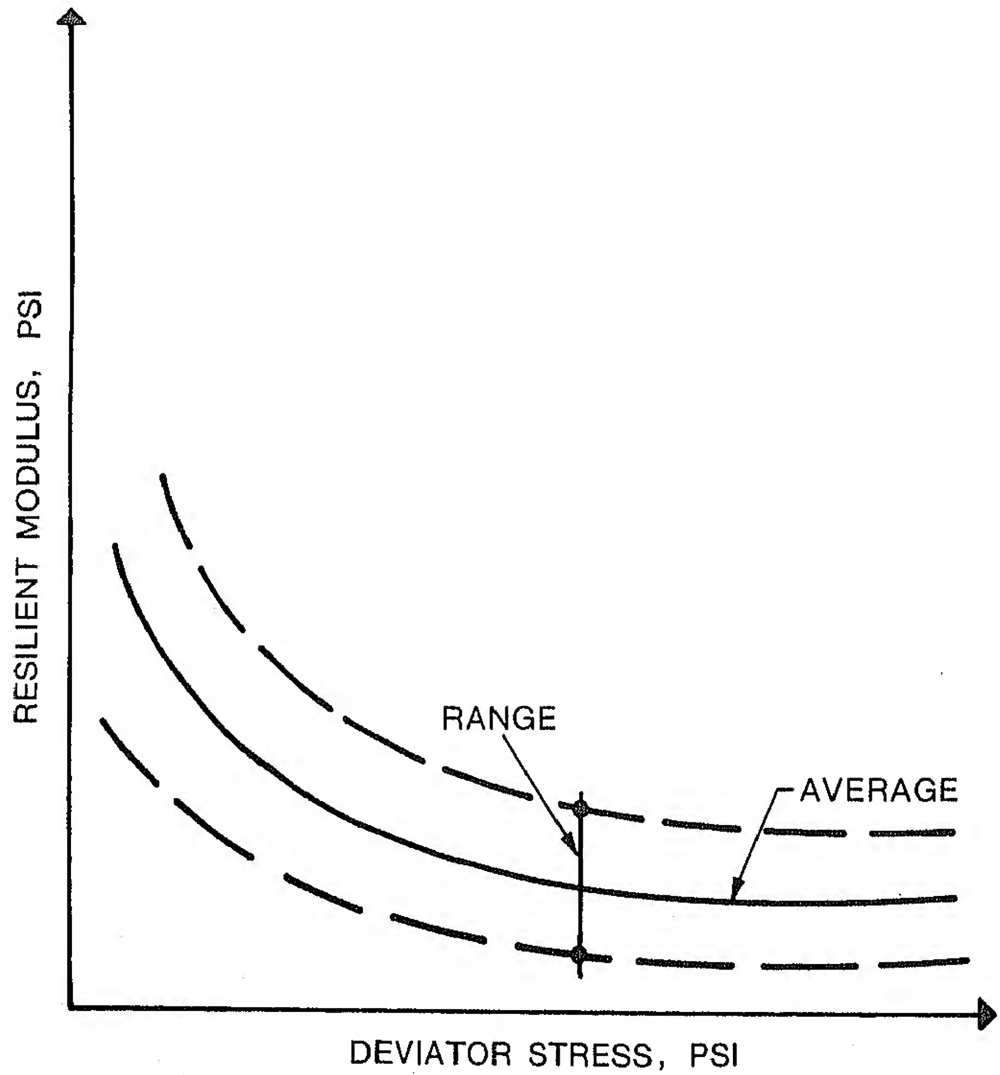


Figure 5-4. Presentation of results of resilience tests on cohesive soils.

3, and 1 psi.

(6) Begin the record test using a confining pressure of 1 psi and an equal value of deviator stress. Record the resilient deformation after 200 repetitions. Increase the deviator stress to twice the confining pressure and record the resilient deformation after 200 repetitions. Repeat until a deviator stress of 4 times the confining pressure is reached (stress ratio of 5).

(7) Repeat as in step 6 above for each value of confining pressure.

(8) When the test is completed, decrease the back pressure to zero, reduce the chamber pressure to zero, and dismantle the cell. Remove the LVDT clamps, etc. Remove the soil specimen, and use the entire amount of soil to determine the moisture content.

b. Calculations can be performed using a similar tabular arrangement as was shown in table 5-1. Test results should be presented in the form of a plot of log M_R versus log of the sum of the principal stresses as shown in figure 5-5.

5-10. Interpretation of test results.

a. As previously indicated, test results for cohesive soils are presented in the form of a plot of resilient modulus (M_R) versus deviator stress (σ_d). Normally for cohesive soils, the test results will indicate that the resilient modulus decreases rapidly with increases in deviator stress. Thus, selection of a resilient modulus from the laboratory tests results requires an estimate of the deviator stress at the top of the subgrade with respect to the design aircraft. For a properly designed pavement, the deviator stress at the top of the subgrade will primarily be a function of the subgrade modulus and the design traffic level. Shown in figure 5-6 are relationships between deviator stress at the top of the subgrade and applicable subgrade modulus values determined from an analysis of the pavement sections described in TM 5-825-2. The relationships shown in figure 5-6 were determined using a layered elastic pavement model with the modulus values as input parameters and the deviator stress values as computed responses. Thus, these relationships are essentially limiting criteria. Relationships are shown for 1,000, 10,000, 100,000, and 1,000,000 repetitions of strain. To determine the appropriate modulus value to use in the performance model, the test results from the resilient modulus tests on the laboratory specimens are superimposed on the appropriate relationship from figure 5-6, and the design modulus value is taken from the intersection of the plotted functions.

b. For example, assume a design problem involving 100,000 repetitions of strain. Figure 5-7 shows a plot of relationship taken from figure 5-6

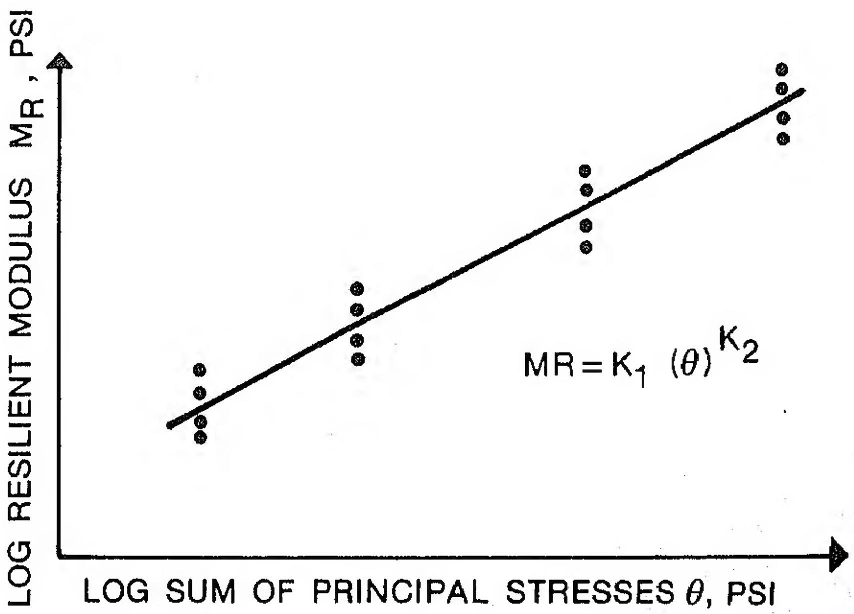


Figure 5-5. Presentation of results of resilience tests on cohesionless soils.

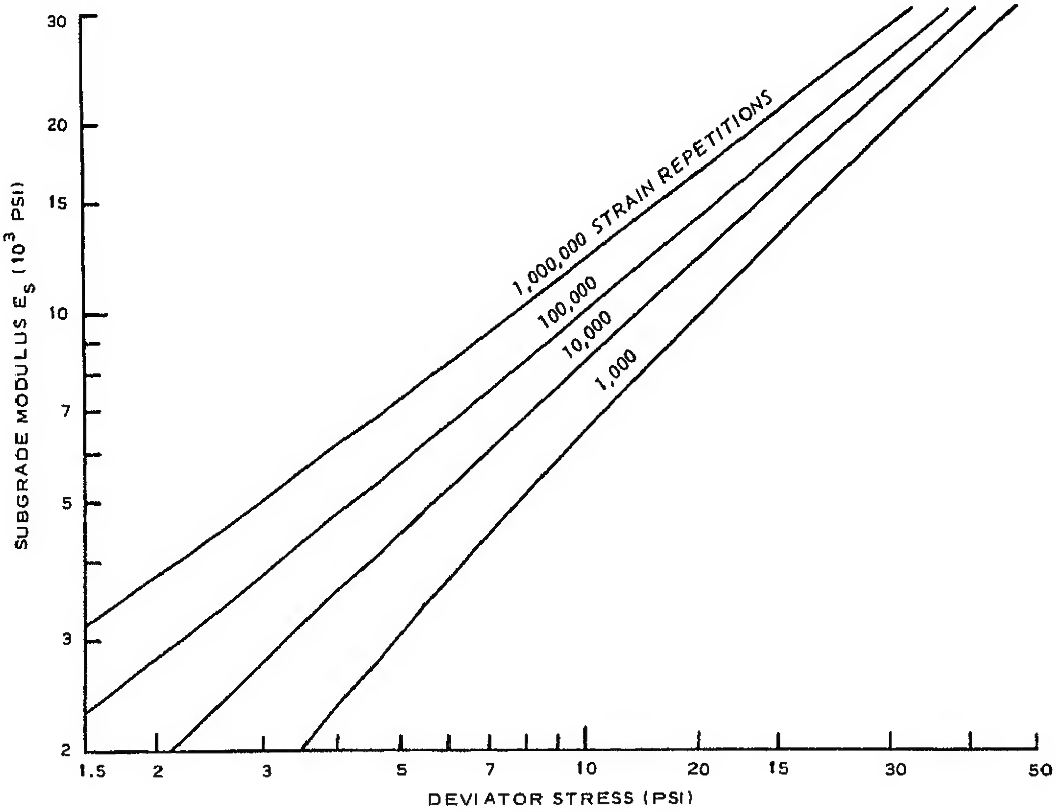


Figure 5-6. Estimated deviator stress at top of subgrade.

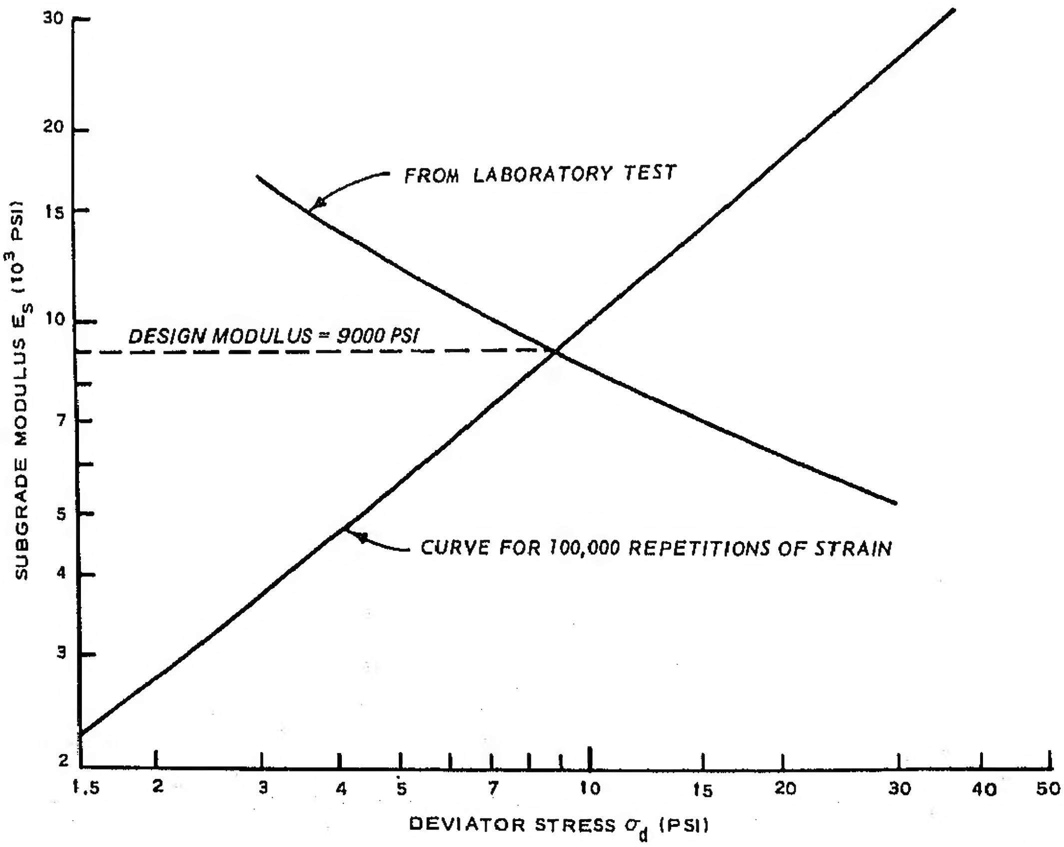


Figure 5-7. Determination of subgrade modulus for cohesive soils.

superimposed on test results from a laboratory resilient modulus test. For this particular design, it can be seen that a subgrade modulus value of 9,000 psi would be used.

c. For cohesionless soils, laboratory test results are presented in the form of a plot of resilient modulus versus the first stress invariant, i.e., sum of the principal stress θ. For cohesionless soils, this relationship is generally linear in form on a log-log plot, with the resilient modulus being directly proportional to the sum of the principal stresses. Selection of a specific resilient modulus value for use in the design model requires an estimate of the sum of the principal stresses at the top of the subgrade. Since a cohesionless material is involved, the influence of both applied stresses and estimated overburden stresses from the pavement structure must be considered. In figure 5-8, a relationship is shown between the pavement thickness and the sum of the principal stresses at the top of the subgrade due to overburden. In figure 5-9, relationships are shown between the subgrade modulus and limiting values of the sum of the principal stresses due to applied force. For each figure, relationships are shown for 1,000, 10,000, 100,000 and 1,000,000 repetitions of stress. Using the value of the estimated pavement thickness, that part of the total sum of the principal stresses due to overburden can be obtained from figure 5-8. The applicable relationship from figure 5-9 is then selected and adjusted to include the influence of overburden by increasing all values of the principal stress sum by the value obtained from figure 5-8. Thus, a new limiting relationship is obtained and replotted. The results of the laboratory modulus test are superimposed on the plot, and the design subgrade modulus values are taken at the intersection of these relationships.

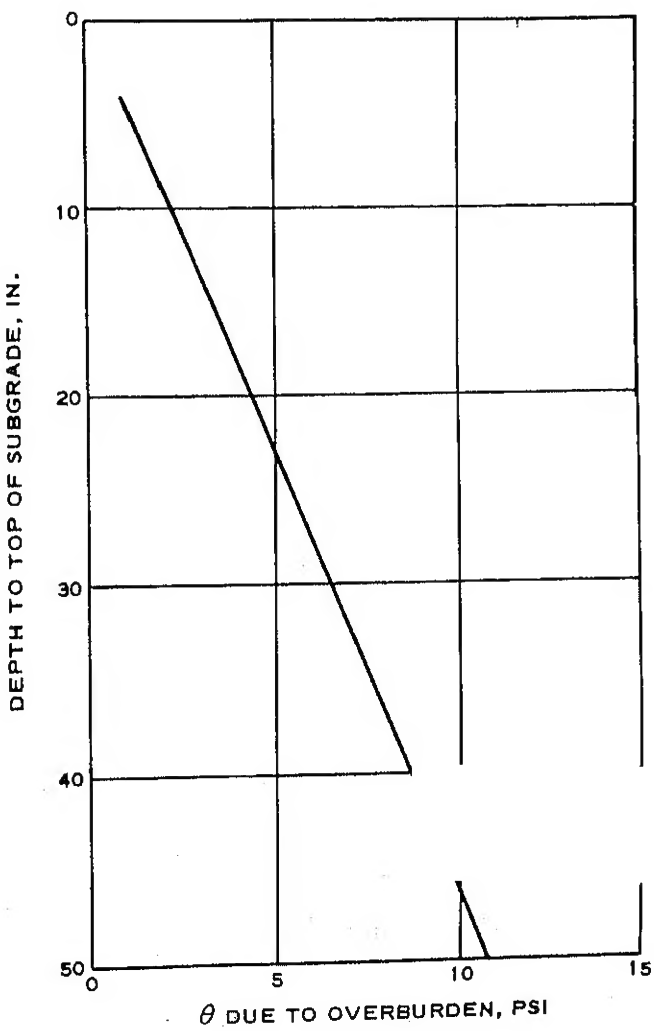


Figure 5-8. Relationship for estimating θ due to overburden.

d. As an example, assume a design problem involving a pavement having an estimated initial thickness of 30 inches. The design aircraft has a dual-wheel main gear assembly, and the design life is for 100,000 repetitions of strain. From figure 5-8, the value of the sum of the principal stresses due to overburden is 6.5 psi. Using the 100,000 strain repetition curve from figure 5-9, the value obtained from figure 5-8 is added to all values of the sum of the principal stresses indicated in the relationship and the adjusted curve is replotted (fig 5-10). The result of adjusting the original relationship is to shift it to the right of its original position. In figure 5-10, the results of laboratory resilient modulus tests on specimens of the subgrade soil are also shown. From the intersection of these two relationships, a design modulus (M_R) of 15,000 psi is determined.

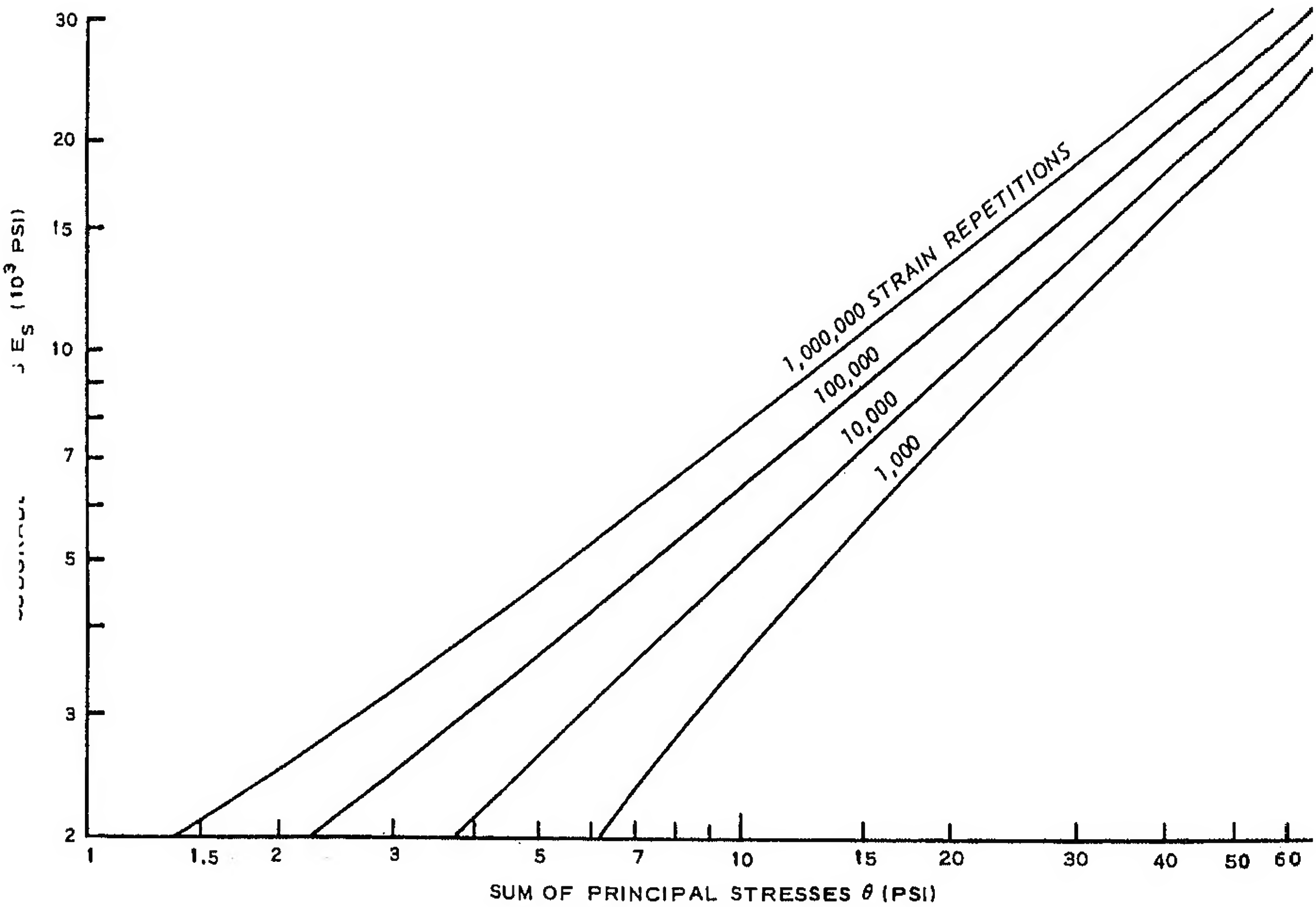


Figure 5-9. Estimated θ at top of subgrade.

e. In some situations, the laboratory curve may not converge with the limiting stress-modulus relationship within the range of values indicated. Obviously, two possibilities are involved in this situation: the laboratory relationships could plot above or below the limiting criteria curve. In the former case, since all values of the sum of the principal stresses indicated by the laboratory curve would exceed the stress criteria within the region under consideration, the value of 30,000 psi should be used for the subgrade modulus. In the latter case, the initial design thickness value should be increased and the limiting criteria curve readjusted until convergence with the laboratory relationships is obtained.

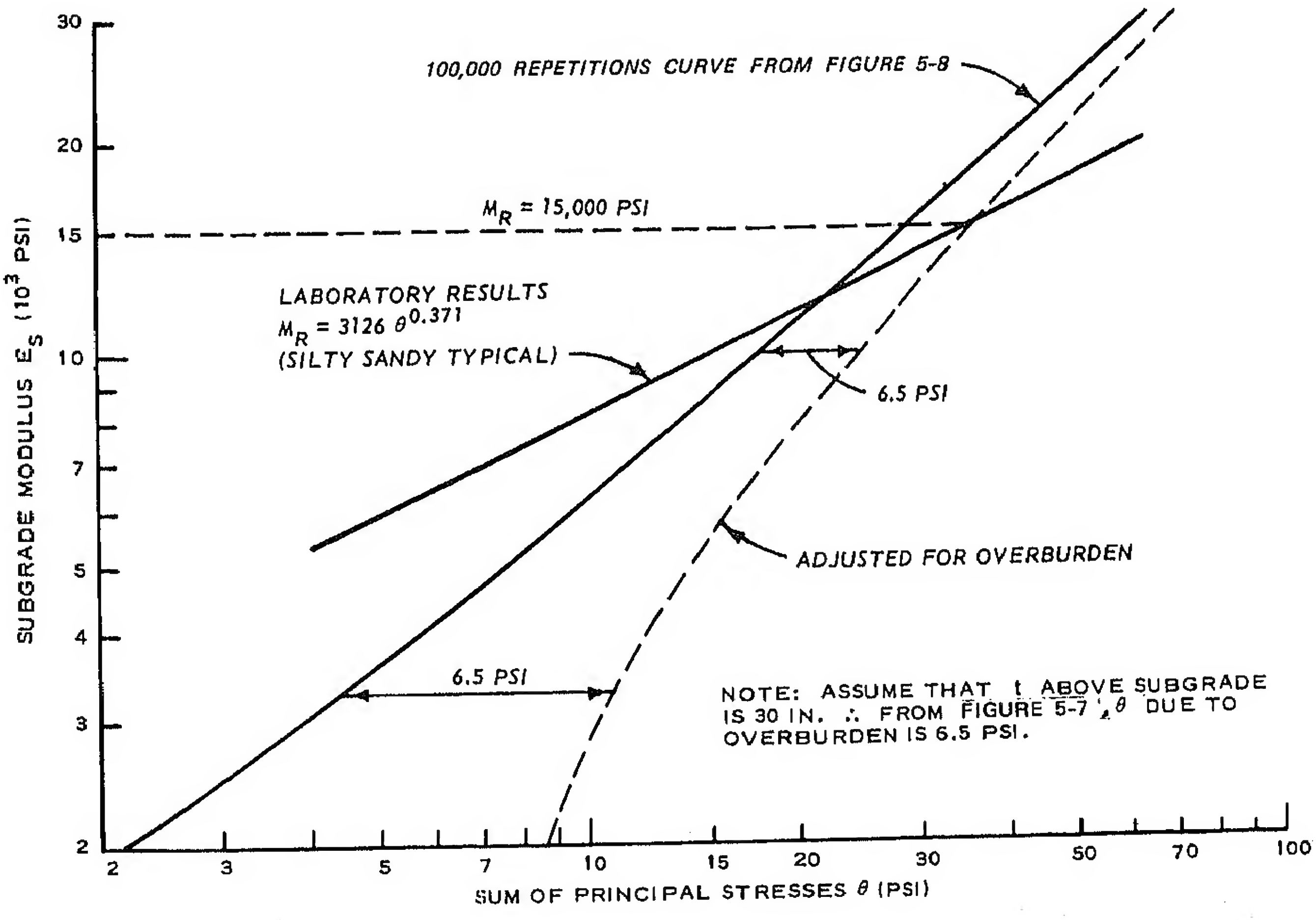


Figure 5-10. Selection of M_R for silty-sand subgrade with estimated thickness of 30 inches for 100,000 repetitions of strain.

CHAPTER 6

PROCEDURE FOR DETERMINING THE MODULUS OF ELASTICITY OF UNBOUND GRANULAR BASE AND SUBBASE COURSE MATERIALS

6-1. Procedure.

a. The procedure is based on relationships developed for the resilient modulus of unbound granular layers as a function of the thickness of the layer and type of material. The modulus relationships are shown in figure 6-1. Modulus values for layer n (the upper layer) are indicated on the ordinate, and those for layer n + 1 (the lower layer) are indicated on the abscissa. Essentially linear relationships are indicated for various thicknesses of base and subbase course materials. For subbase courses, relationships are shown for thicknesses of 4, 5, 6, 7, and 8 inches. For subbase courses having a design thickness of 8 inches or less, the applicable curve or appropriate interpolation can be used directly. For a design subbase course thickness in excess of 8 inches, the layer should be divided into sublayers of approximately equal thickness and the modulus of each sublayer determined in-

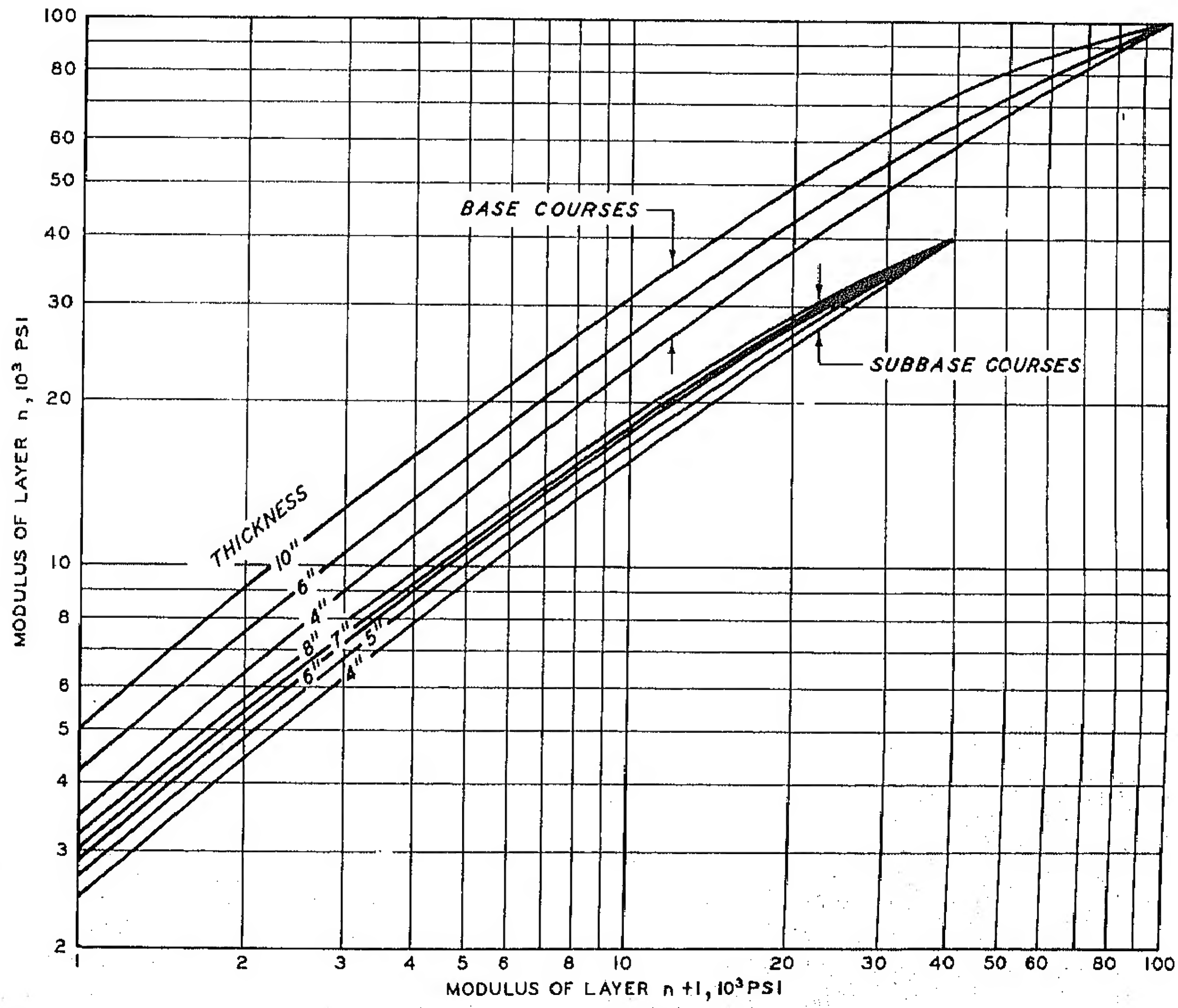


Figure 6-1. Relationships between modulus of layer n and modulus of layer n + 1 for various thicknesses of unbound base course and subbase course.

dividually. For base courses, relationships are shown for thicknesses of 4, 6, and 10 inches. These relationships can be used directly or by interpolation for design base course thicknesses up to 10 inches. For design thicknesses in excess of 10 inches, the layer should also be divided into sublayers of approximately equal thickness and the modulus of each sublayer determined individually.

b. To determine modulus values from this procedure, figure 6-1 is entered along the abscissa using modulus values of the subgrade or underlying layer (modulus of layer n + 1). At the intersection of the curve applicable to this value with the appropriate thickness relationship, the value of the modulus of the overlying layer is read from the ordinate (modulus of layer n). This procedure is repeated using the modulus value just determined as the modulus of layer n + 1 to determine the modulus value of the next overlying layer.

6-2. Examples.

a. Assume a pavement having a base course thickness of 4 inches and a subbase course thickness of 8 inches over a subgrade having a modulus of 10,000 psi. Initially, the subgrade is assumed to be layer n + 1 and the subbase course to be layer n. Entering figure 6-1 with a modulus of layer n + 1 of 10,000 psi and using the 8-inch subbase course curve, the modulus of the subbase (layer n) is found to be 18,500 psi. In order to determine the modulus value of the base course, the subbase course is now assumed to be layer n + 1 and the base course to be layer n. Entering figure 6-1 with a modulus value of layer n + 1 of 18,500 psi and using the 4-inch base course relationship, the modulus of the base course is found to be 36,000 psi. Modulus values determined for each layer are indicated in figure 6-2.

b. If, in the first example, the design thickness of the subbase course had been 12 inches, it would have been necessary to divide this layer into two 6-inch-thick sublayers. Then, using the procedure described above for the second example, the modulus values determined for the lower and upper sublayers of the subbase course and for the base course are 17,500, 25,500, and 44,000 psi respectively. These values are shown in figure 6-3.

c. The relationships indicated in figure 6-1 can be expressed as

$$E_n = E_{n+1} (1 + 10.52 \log t - 2.10 \log E_{n+1} \log t)$$

where

n = a layer in the pavement system

E_n = resilient modulus (in psi) of layer n

E_{n+1} = the resilient modulus (in psi) of the layer beneath layer n

t = the thickness (in psi) of layer n

for base course materials and as

$$E_n = E_{n+1} (1 + 7.18 \log t - 1.56 \log E_{n+1} \log t)$$

for subbase course materials. Use of these equations for direct computation of modulus values for the examples given above yields the value

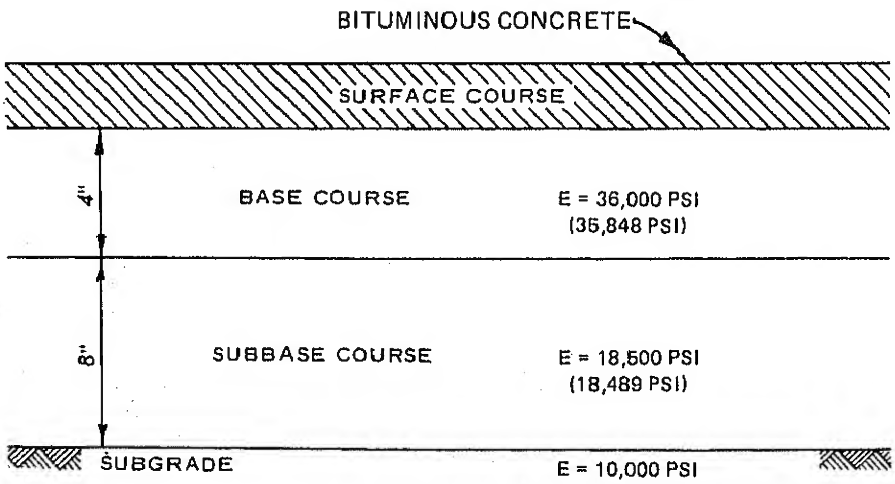


Figure 6-2. Modulus values determined for first example.

indicated in parentheses in figures 6-2 and 6-3. It can be seen that comparable values are obtained with either graphical or computational determination of the modulus value for either material.

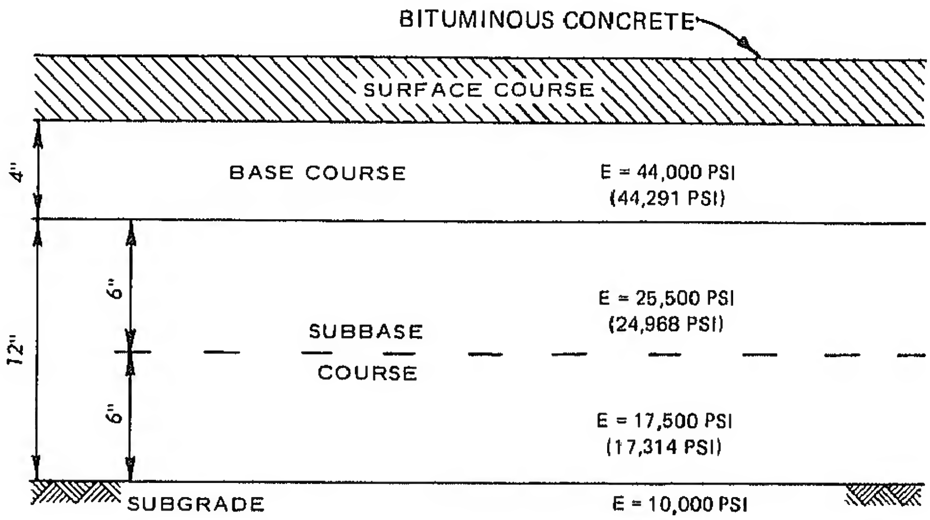


Figure 6-3. Modulus values determined for second example.

CHAPTER 7

PROCEDURES FOR DETERMINING THE FLEXURAL MODULUS AND FATIGUE CHARACTERISTICS OF STABILIZED SOILS

7-1. Laboratory procedure.

a. General. The procedure involves application of a repetitive loading to a laboratory-prepared beam specimen under controlled stress conditions. Applied load and deflection along the neutral axis and at the lower surface are monitored, and the results are used to determine the flexural modulus and fatigue characteristics.

b. Specimen preparation. Beam specimens should be prepared following the general procedures indicated in American Society for Testing and Materials (ASTM) D 1632. This method describes procedures for molding 3- by 3- by 11-1/4-inch specimens; however, any size mold may be used for the test. For soils containing aggregate particles larger than 3/4 inch, it is recommended that molds on the order of 4 by 4 to 6 by 6 inches be used. In general, specimens should have an approximately square cross-sectional configuration and a length adequate to accommodate an effective test span equal to three times the height or width. Specimens should be molded to the stabilizer treatment level, moisture content, and density expected in the field structures. Cement-treated materials should be moist-cured for 7 days. Lime-treated materials should be cured for 28 days at 73 degrees F.

c. Equipment. The following equipment is required:

— Loading frame capable of receiving specimen for third-point loading test.
— Electrohydraulic testing machine. This machine must be capable of applying static and haversine loads.
— Load cell (approximately 2,000-pound capacity).
— Two LVDT's and one SR-4 type strain gage.
— Recording equipment for monitoring deflection, strain, and load.
— Miscellaneous pins and yokes, as described in the equipment setup below, for mounting the LVDT's.

d. Equipment setup. Details of the equipment setup are shown in figures 7-1 to 7-3. The beam should be positioned so that the molding lamina-

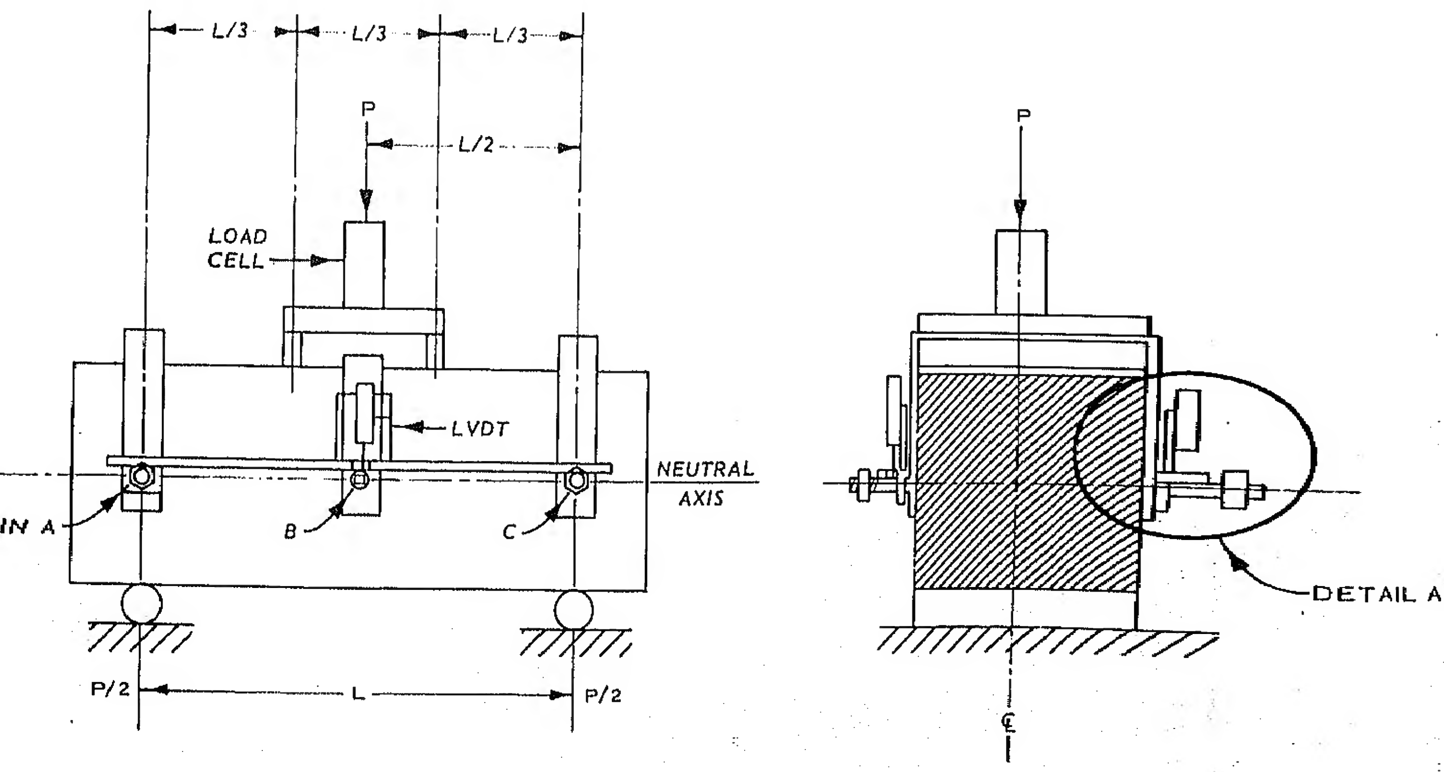


Figure 7-1. General view of equipment setup.

tions are horizontal. The three yokes are positioned over the top of the beam and held in place by threaded pins positioned along the neutral axis. The end pins, pins A and C, are positioned directly over the end reaction points, and the middle pin, pin B, is positioned at the center of the beam. A metal bar rests on top of the pin. At the A position, the bar is equipped with a lower vertical tab having a hole that slips loosely over the pin. A nut is placed on the end of the pin to prevent the bar from slipping. At the center or B position, the bar is equipped with a vertical tap onto which an LVDT is cemented in a vertical position. At this point on the bar, there is a hole through which the LVDT core pin falls to rest on the B pin. This pin must be fabricated with flat sides on the shaft to provide a horizontal surface on which the LVDT core pin rests. At the C position, the end of the bar simply rests on the unthreaded portion of the C pin. A nut is placed on the end of the C pin to prevent excessive side movement of the bar end. This type of bar, pin, and LVDT arrangement is provided on both sides of the beam. Although no dimensions are provided in figures 7-1 to 7-3, this type of equipment can easily be dimensioned and fabricated to fit any size beam. Either steel or aluminum may be used. The beam should be positioned and arranged to accommodate third-point loading as indicated in figure 7-2. As the beam bends under loading, deflection at the center is measured by determining the movement of the LVDT stems from their original positions. The LVDT's are connected to the monitoring system to give an average deflection reading. Since it is also desired to determine the maximum tensile strain of the beam under loading, an SR-4 strain gage should be attached to the lower beam surface with epoxy or some other suitable cement and should also be connected to the monitoring system. If it is not possible to determine strain directly, a strain value may be found using equation 7-2.

e. Test procedure. The flexural beam test is a stress-controlled test. Therefore, an intial specimen should be statically loaded to failure, and the stress level for the initial repetitive load tests should be set at 50 percent of the maximum rupture load. The repetitive load test should be conducted using a haversine wave form, a loading duration of 0.5 second, and a frequency of about 1 hertz. To develop a strain repetition pattern, it is recommended that tests be conducted at 40, 50, 60, and 70 percent of the maximum rupture value; however, stress levels can be varied to higher or lower levels. Data to be monitored include load, deflection along the neutral axis, strain at the lower surface of the specimen, and number of repetitions.

f. Reporting of test results.

(1) Flexural modulus. The flexural modulus should be determined at 100, 1,000, and 10,000 load

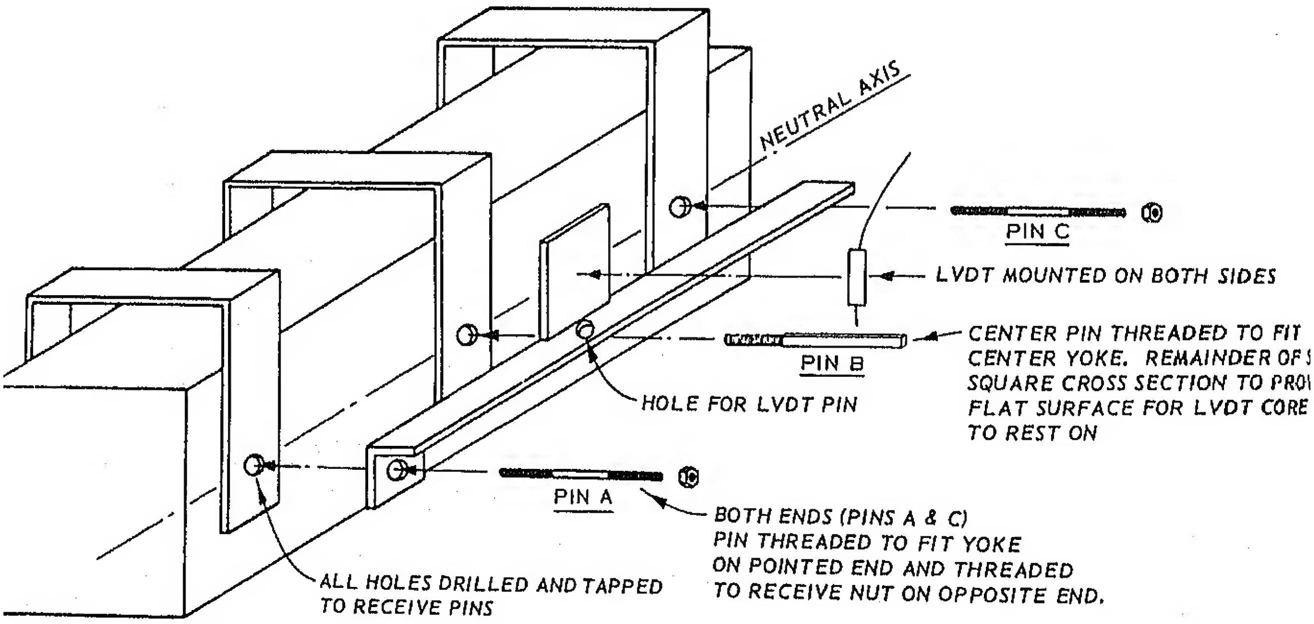


Figure 7-2. Details of equipment setup.

repetitions or at failure. This value may be determined from load deflection data monitored at these repetition levels using the expression

$$E_f = \frac{23PL^3}{1296dI}\left\{1 + 2.11\left(\frac{h}{L}\right)^2\right\} \qquad \text{(eq 7-1)}$$

where

E_f = flexural modulus, psi
P = maximum load amplitude, pounds
L = specimen length, inches
d = deflection at the neutral axis, inches
I = moment of inertia, the square of inches squared
h = specimen height, inches

The value to be used for E_f in the performance model is the arithmetic mean of all values obtained during the test.

(2) *Fatigue characteristics.* Fatigue characteristics are presented as a plot of strain indicated at the bottom surface of the specimen versus load repetitions at failure. Generally, the value of the strain obtained during the first few load repetitions is the value to be plotted. If no direct means of measuring strain is available, a strain value ϵ may be computed using the expression

$$\epsilon = \frac{PLh}{6E_f I} \qquad \text{(eq 7-2)}$$

7-2. Graphical determination of flexural modulus for chemically stabilized soils (cracked section).

The procedure for determining a flexural modulus value for chemically stabilized soils based on the cracked section concept involves the use of a relationship between unconfined compressive strength and flexural modulus determined analytically. This relationship is shown in figure 3-2. To use this relationship, specimens of the stabilized material should be molded and tested following procedures indicated in ASTM D 1633. Values obtained from the unconfined compression test can then be used to determine the values of the equivalent cracked section modulus using figure 3-2.

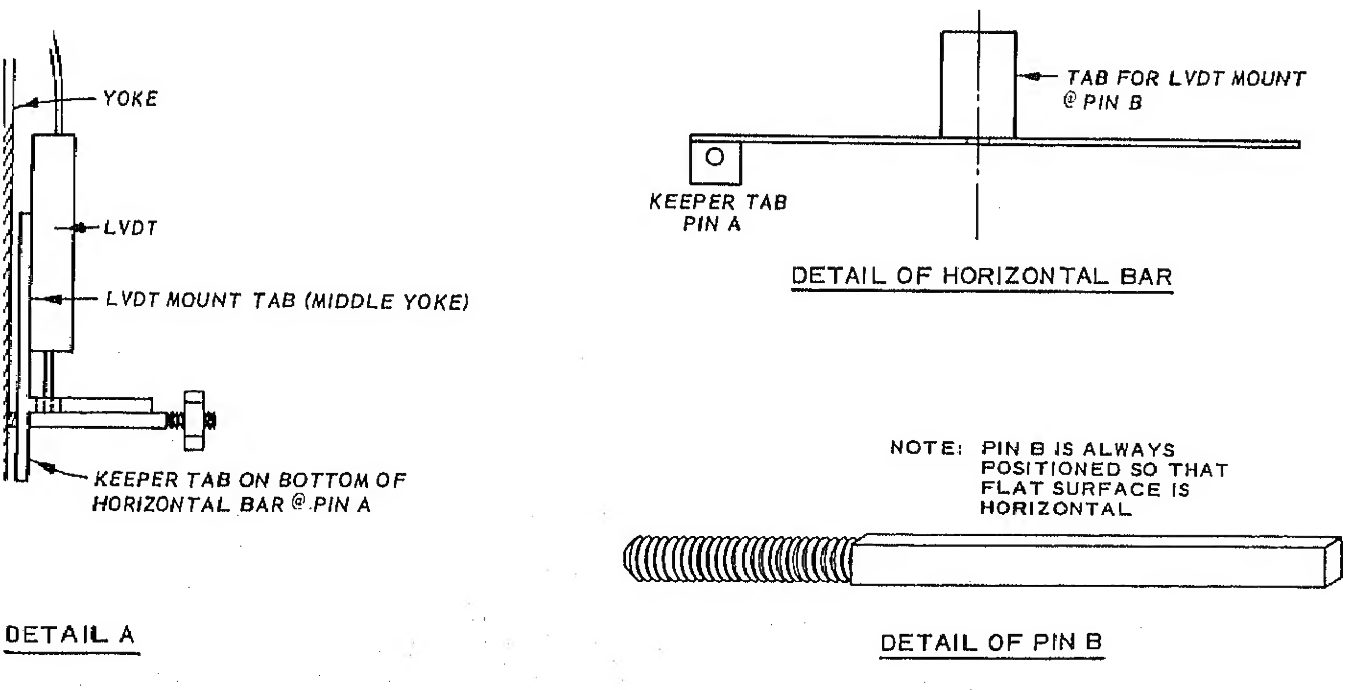


Figure 7-3. Miscellaneous details.

CHAPTER 8

PROCEDURE FOR PREPARATION OF BITUMINOUS CYLINDRICAL SPECIMENS

8-1. Scope.

This procedure describes the preparation of cylindrical specimens of bituminous paving mixture suitable for dynamic modulus testing. The procedure is intended for dense-graded bituminous concrete mixture containing up to 1-inch maximum-size aggregate.

8-2. Applicable standards.

The following ASTM publications are applicable to this procedure: D 1559, D 1560, and D 1561.

8-3. Specimens.

Approximately 4,000 grams of bituminous mixture should be prepared as specified by ASTM D 1560. Cylindrical specimens should be 4 inches in diameter by 8 inches in height.

8-4. Apparatus.

a. The apparatus used in preparing the specimens should be as specified by ASTM D 1561, except that steel molding cylinders with 1/4-inch wall thickness having an inside diameter of 4 inches and height of 10 inches should be used.

b. The measurement system should consist of a two-channel recorder, stress and strain measuring devices, and suitable signal amplification and excitation equipment. The measurement system should have the capability for determining loading up to 3,000 pounds from a recording with a minimum sensitivity of 2 percent of the test load per millimeter of chart paper. This system should also be capable of use in determining strains over a range of full-scale recorder outputs from 300 to 5,000 microunits of strain. At the highest sensitivity setting, the system should be able to display 4 microunits of strain or less per millimeter on the recorder chart.

c. The recorder amplitude should be independent of frequency for tests conducted up to 20 hertz.

d. The values of axial strain should be measured by bonding two wire strain gages at midheight opposite each other on the specimens. (The Baldwin Lima Hamilton SR-4 Type A-1S 13 strain gage has been found satisfactory for this purpose.) The gages are wired in a Wheatstone bridge circuit with two active gages on the test specimen exposed to the same environment as the test specimen. The temperature-compensating gages should be at the same position on the specimen as the active gages. The sensitivity and type of measurement device should be selected to provide the strain readout required above.

e. Loads should be measured with an electronic load cell meeting requirements for load and stress measurements above.

8-5. Procedure.

a. The compaction temperature for the bituminous mixture should be as specified by ASTM D 1561. As the first step in molding specimens, heat the compaction mold to the same temperature as the mix. Next, place the compaction mold in position in the mold holder and insert a paper disk 4 inches in diameter to cover the baseplate of the mold holder. Weigh out one-half of the required amount of bituminous mixture for one specimen at the specified temperature and place uniformly in the insulated feeder trough, which has been preheated to the compaction temperature for the mixture. By means of the variable transformer controlling the heater, maintain the compactor foot sufficiently hot to prevent the mixture from adhering to it. By means of a paddle of suitable dimensions to fit the cross section of the trough, push 30 approximately equal portions of the mixture continuously and uniformly into the mold while 30 tamping blows at a pressure of 250 psi are applied. Immediately place the remaining one-half of the mixture uniformly in the feeder trough. Push 30 approximately equal portions of the mixture continuously and uniformly into the mold while 30 tamping blows at a pressure of 250 psi are applied. If sandy or unstable material is involved and there is undue movement of the mixture under the compactor foot, reduce the compaction temperature and compactor foot pressure until kneading compaction can be accomplished.

b. Immediately after compaction with the California kneading compactor, apply a static load to the specimen using a compression testing machine. Apply the load by the double-plunger method in which metal followers are employed as free-fitting plungers on the top and bottom of the specimen. Apply the load on the specimen at a rate of 0.5 inches per minute until an applied pressure

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b. Immediately after compaction with the California kneading compactor, apply a static load to the specimen using a compression testing machine. Apply the load by the double-plunger method in which metal followers are employed as free-fitting plungers on the top and bottom of the specimen. Apply the load on the specimen at a rate of 0.5 inches per minute until an applied pressure

of 1,000 psi is reached. Release the load immediately. After the compacted specimen has cooled sufficiently so that it will not deform on handling, remove it from the mold. Place the specimen on a smooth flat surface and allow to cool to room temperature. Cylindrical specimens will have approximately the same bulk specific gravity as specimens prepared as specified by ASTM D 1559 and ASTM D 1561.

CHAPTER 9

LABORATORY PROCEDURE FOR DETERMINING THE DYNAMIC MODULUS OF BITUMINOUS CONCRETE MIXTURES

9-1. General.

The purpose of this procedure is to determine dynamic modulus values of bituminous concrete mixtures. The procedure described covers a range of both temperature and loading frequency. The minimum recommended test series consists of testing at 40, 70, and 100 degrees F. at loading frequencies of 2 and 10 hertz for each temperature. The method is applicable to bituminous paving mixtures similar to the 1-, 3/4-, 1/2-, and 3/8-inch, and No. 4 mixes as defined by Table 3 of ASTM D 3515.

9-2. Applicable standards.

The following ASTM standards are applicable to this procedure: C 617, D 1559, D 1561, and D 3515.

9-3. Summary Procedure.

The dynamic modulus test is run by applying a sinusoidal (haversine) axial compressive stress to a specimen of bituminous concrete at a given temperature and loading frequency. The resulting recoverable axial strain response of the specimen is measured and used to calculate the dynamic modulus.

9-4. Definitions.

The following terms are used in this procedure:

a. Dynamic modulus. The absolute value of the complex modulus which defines the elastic properties of a linear viscoelastic material subjected to a sinusoidal loading.

b. Complex modulus. A complex number which defines the relationship between stress and strain for a linear viscoelastic material.

c. Linear material. A material whose stress-to-strain ratio is independent of the loading stress applied.

9-5. Apparatus.

a. An electrohydraulic testing machine with a frequency generator capable of producing a haversine wave form has proven to be most suitable for use in dynamic modulus testing. The testing machine should have the capability of applying loads over a range of frequencies from 1 to 20 hertz and stress levels up to 100 psi.

b. The temperature control system should be capable of a temperature range of 32 to 120 degrees F. The temperature chamber should be large enough to hold six specimens.

c. A hardened steel disk with a diameter equal to that of the test specimen should be used to transfer the load from the testing machine to the specimen.

9-6. Specimens.

The laboratory-molded specimens should be prepared according to chapter 8. A minimum of three specimens is required for testing. The molding procedure is as follows: Cap all specimens with a sulfur mortar meeting ASTM C 617 requirements prior to testing. Bond the strain gages with epoxy cement to the sides of the specimen near midheight in position to measure axial strains. (Baldwin Lima Hamilton EPY 150 Epoxy Cement has been found satisfactory for this purpose. On specimens with large-size aggregate, care must be taken so that the gages are attached over areas between the aggregate faces.) Wire the strain gages, as required in paragraph 9-5 above, and attach suitable lead wires and connectors.

9-7. Procedure.

a. Place test specimens in a controlled temperature cabinet, and bring them to the specified test temperature. A dummy specimen with a thermocouple in the center can be used to determine when the desired test temperature is reached.

b. Place a specimen in the loading apparatus, and connect the strain gage wires to the measurement system. Put the hardened steel disk on top of the specimen and center both under the loading apparatus. Adjust and balance the electronic measuring system as necessary.

c. Apply the haversine loading to the specimen without impact and with loads varying between 0 and 35 psi for each load application for a minimum of 30 seconds and not exceeding 45 seconds at temperature of 40, 70, and 100 degrees F. and at loading frequencies of 2 hertz for taxiway design and 10 hertz for runway design. If excessive deformation (greater than 2,500 microunits of strain) occurs, reduce the maximum loading stress level to 17.5 psi.

d. Test three specimens at each temperature and frequency condition twice. Start at the lowest temperature and repeat the test at the next highest

temperature. Bring the specimens to the specified test temperature before each test is commenced.

e. Monitor both the loading stress and the axial strain during the test. Increase the recorder chart speed so that one cycle covers 1 to 2 centimetres of chart paper for five to ten repetitions before the end of the test.

f. Complete the loading for each test within 2 minutes from the time specimens are removed from the temperature control cabinet. The 2-minute testing time limit is waived if loading is conducted within a temperature control cabinet meeting requirements in paragraph 9-5.

9-8. Calculations.

Measure the average amplitude of the load and the strain over the last three loading cycles to the nearest 1/2 millimeter. Calculate the loading stress (σ_o) using the equation

$$\sigma_o = \frac{H_1 L}{H_2 A} \qquad \text{(eq 9-1)}$$

where

H_1 = measured height of load

H_2 = measured chart height

L = full-scale load amplitude determined by settings on the recording equipment

A = cross-section area of the test specimen

Calculate the recoverable axial strain (ϵ_o) using the equation

$$\epsilon_o = \frac{H_3 S}{H_4} \qquad \text{(eq 9-2)}$$

where

H_3 = measured height of recoverable strain

H_4 = measured chart height

S = full-scale strain amplitude determined by settings on the recording equipment

Calculate the dynamic modulus ($|E^*|$) using the equation

$$|E^*| = \frac{\sigma_o}{\epsilon_o} \qquad \text{(eq 9-3)}$$

where

σ_o = axial loading stress, psi

ϵ_o = recoverable axial strain, inches per inch

Report the average dynamic modulus at temperatures of 40, 70, and 100 degrees F. for each loading frequency at each temperature.

CHAPTER 10

PROCEDURE FOR ESTIMATING THE MODULUS OF ELASTICITY OF BITUMINOUS CONCRETE

10-1. General.

The procedure for estimating the modulus of elasticity of bituminous concrete presented here is based on relationships developed by Shell.* Parameters needed for input into this method are:

— Ring-and-ball softening point, in degrees F., of the bituminous material used in the mix in accordance with ASTM D 36.

— Penetration of the bituminous material, in 1/10 millimeters, in accordance with ASTM D 5.

— Volume concentration of the aggregate (C_v) used in the mix defined by

$$C_v = \frac{\text{aggregate volume}}{\text{aggregate volume} + \text{bitumen volume}} \quad \text{(eq 10-1)}$$

10-2. Steps of procedure.

The steps in using this method are as follows:

a. With known values of penetration and ring-and-ball softening point, enter figure 10-1 and determine the penetration index (PI).

b. The next step involves the use of the nomograph presented in figure 10-2. In addition to the PI, two other values are required: the temperature of the bituminous concrete mix for which the modulus value is desired, and the estimated loading frequency or time of loading to which the prototype pavement will be subjected. Use of a loading frequency of 2 hertz is recommended for taxiway design and 10 hertz for runway design. With values for the loading frequency and the difference in temperature between the bituminous concrete and the ring-and-ball softening point, a stiffness value for the bitumen S_{bit} can be determined from the appropriate PI line at the top of the nomograph. The value of S_{bit} is then used to determine to modulus of the mix S_{mix}.

c. A value for S_{mix} may be determined by

$$S_{mix} = S_{bit}\left[1 + \left(\frac{2.5}{n}\right)\left(\frac{C_v}{1 - C_v}\right)\right]^n \quad \text{(eq 10-2)}$$

where

$$n = 0.83 \log\left(\frac{400{,}000}{S_{bit}}\right) \quad \text{(eq 10-3)}$$

The value thus determined for S_{mix} is in units of kilograms per square centimeter.

d. This expression should be used for aggregate volume centrations of 0.7 to 0.9 and air void contents of 3 percent or less. For larger air void contents, use a corrected aggregate volume concentration (C_v')

$$C_v' = \frac{C_v}{1 + \Delta\text{air void content}} \quad \text{(eq 10-4)}$$

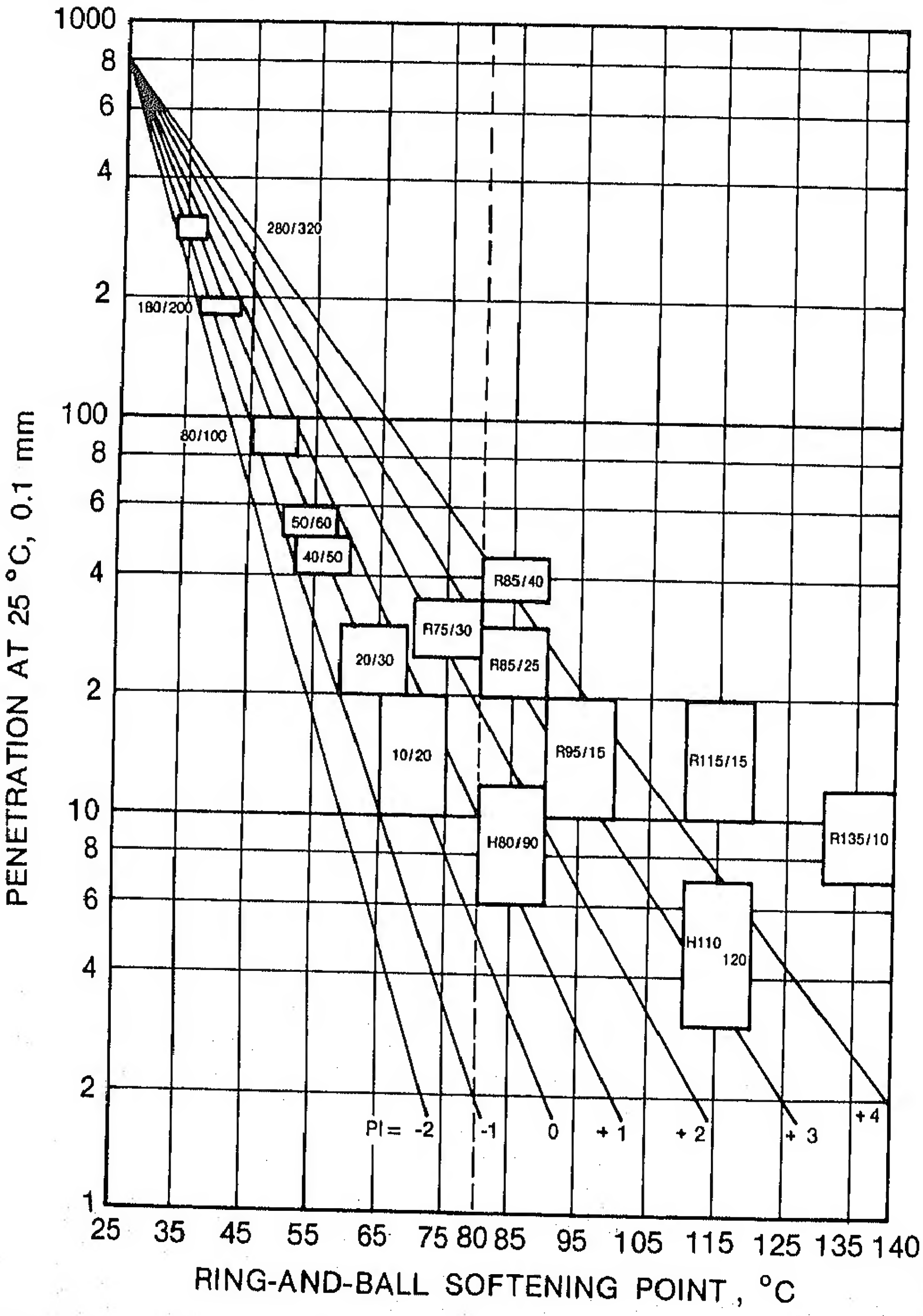
where Δair void content is the actual air void content (expressed in decimal form) minus 0.03. Equation 10-4 is valid only when

* Heukelom, W. and Klomp, A. J. G., "Road Design and Dynamic Loading," *Proceedings, Association of Asphalt Paving Technologists,* Vol 33, 1964, pp 92-125.

$$C_B \geq \frac{2}{3}(1 - C_V) \qquad \text{(eq 10-5)}$$

where

$$C_B = \frac{\text{bitumen volume}}{\text{aggregate volume + bitumen volume}} \qquad \text{(eq 10-6)}$$



10-1. Relationship between penetration at 25 degrees C. and ring-and-ball softening point for bitumens with different PI's.

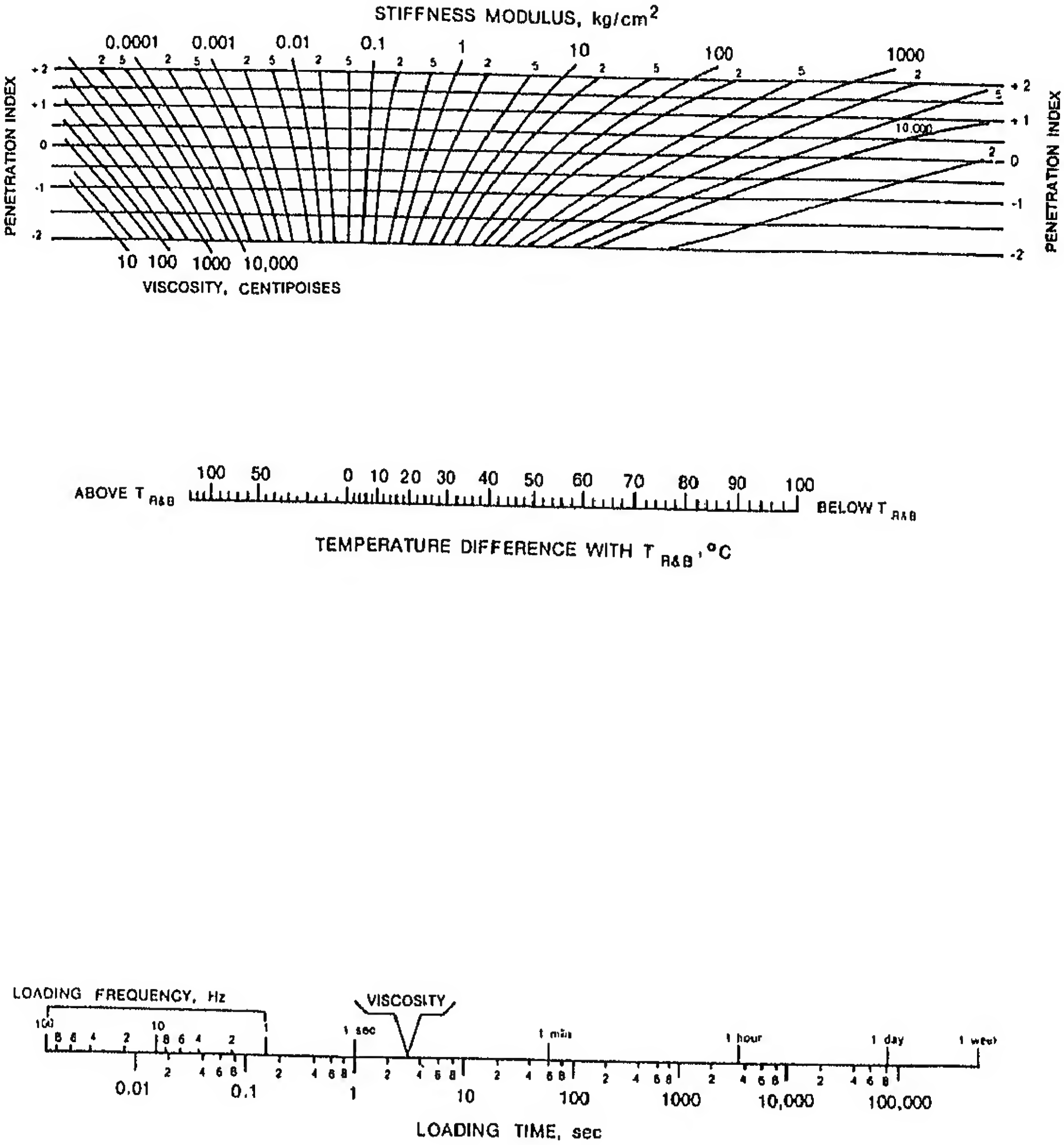


Figure 10-2. Nomograph for determining the stiffness modulus of bitumens.

CHAPTER 11

PROCEDURES FOR DETERMINING THE FATIGUE LIFE OF BITUMINOUS CONCRETE

11-3. Laboratory test method.

a. General. A laboratory procedure for determining the fatigue life of bituminous concrete paving mixtures containing aggregate with maximum sizes up to 1-1/2 inches is described in this chapter. The fatigue life of a simply supported beam specimen subjected to third-point loadings applied during controlled stress-mode flexural fatigue tests is determined.

b. Definitions. The following symbols are used in the description of this procedure:

(1) ϵ = initial extreme fiber strain (tensile and compressive, inches per inch.

(2) N_f = fatigue life of the specimen, number of load repetitions to fracture.

Extreme fiber strain of simply supported beam specimens subjected to third-point loadings, which produces uniaxial bending stresses, is calculated from

$$\epsilon = \frac{12td}{(3L^2 - 4a^2)} \qquad \text{(eq 11-1)}$$

where

t = specimen depth, inches
d = dynamic deflection of beam center, inches
L = reaction span length, inches
a = L/3, inches

c. Test equipment.

(1) The repeated flexure apparatus is shown in figure 11-1. It accommodates beam specimens 15 inches long with widths and depths not exceeding 3 inches. A 3,000-pound-capacity electrohydraulic testing machine capable of applying repeated tension-compression loads in the form of haversine waves for 0.1-second durations with 0.4-second rest periods is used for flexural fatigue tests. Any dynamic testing machine or pneumatic pressure

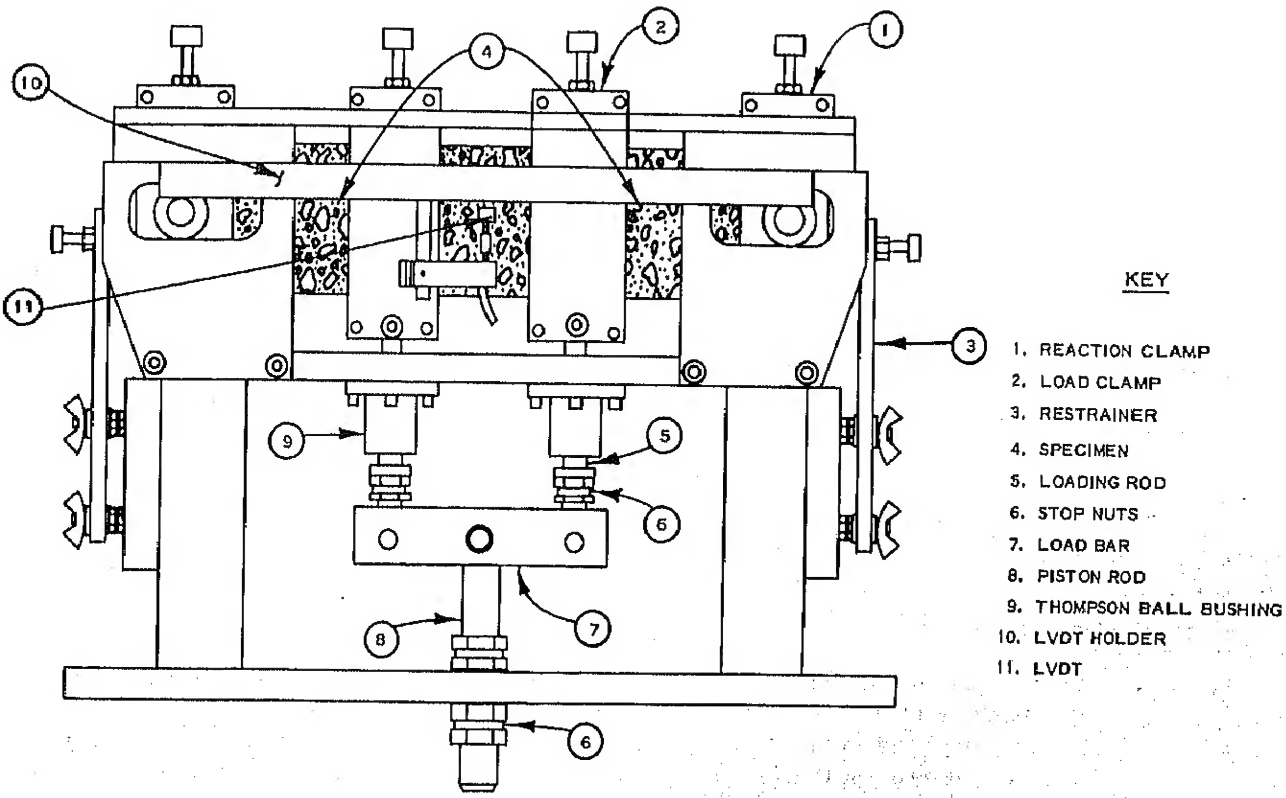


Figure 11-1. Repeated flexure apparatus.

system with similar loading capabilities is also suitable. Third-point loading, i.e., loads applied at distances of L/3 from the reaction points, produces an approximately constant bending moment over the center 4 inches of a 15-inch-long beam specimen with widths and depths not exceeding 3 inches. A sufficient load, approximately 10 percent of the load deflecting the beam upward, is applied in the opposite direction, forcing the beam to return to its original horizontal position and holding it at that position during the rest period. Adjustable stop nuts installed on the flexure apparatus loading rod prevent the beam from bending below the initial horizontal position during the rest period.

(2) The dynamic deflection of the beam's center is measured with an LVDT. An LVDT that has been found suitable for this purpose is the Sheavitz type 100 M-L. The LVDT core is attached to a nut bonded with epoxy cement to the center of the specimen. Outputs of the LVDT and the electrohydraulic testing machine's load cell, through which loads are applied and controlled, can be fed to any suitable recorder. The repeated flexure apparatus is enclosed in a controlled-temperature cabinet capable of controlling temperatures within ±1/2 degree F. A Missimer's model 100 × 500 carbon dioxide plug-in temperature conditioner has been found to provide suitable temperature control.

d. Specimen preparation. Beam specimens 15 nches long with 3-1/2-inch depths and 3-1/4-inch widths are prepared according to ASTM D 3202. If there is undue movement of the mixture under the compactor foot during beam compaction, the temperature, foot pressure, and number of tamping blows should be reduced. Similar modifications to compaction procedures should be made if specimens with less density are desired. A diamond-blade masonry saw is used to cut 3-inch or slightly less deep by 3-inch or slightly less wide test specimens from the 15-inch-long beams. Specimens with suitable dimensions can also be cut from pavement samples. The widths and depths of the specimens are measured to the nearest 0.01 inch at the center and at 2 inches from both sides of the center. Mean values are determined and used for subsequent calculations.

e. Test procedures.

(1) Repeated flexure apparatus loading clamps are adjusted to the same level as the reaction lamps. The specimen is clamped in the fixture sing a jig to position the centers of the two loading lamps 2 inches from the beam center and to osition the centers of the two reactions clamps 6-/2 inches from the beam center. Double layers of Teflon sheets are placed between the specimen and the loading clamps to reduce friction and longitudinal restraint caused by the clamps.

(2) After the beam has reached the desired test temperature, repeated loads are applied. Duration of a load repetition is 0.1 second with 0.4-second rest periods between loads. The applied load should be that which produces an extreme fiber stress level suitable for flexural fatigue tests. For fatigue tests on typical bituminous concrete paving mixtures, the following ranges of extreme fiber stress levels are suggested:

Temperature, degrees F.	*Stress Level Range, psi*
55	150 to 450
70	75 to 300
85	35 to 200

The beam center point deflection and applied dynamic load are measured immediately after 200 load repetitions for calculation of extreme fiber strain ϵ. The test is continued at the constant stress level until the specimen fractures. The apparatus and procedures described have been found suitable for flexural fatigue tests at temperatures ranging from 40 to 100 degrees F. and for extreme fiber stress levels up to 450 psi. Extreme fiber stress levels for flexural fatigue tests at any temperature should not exceed that which causes specimen fracture before at least 1,000 load repetitions are applied.

(3) A set of 8 to 12 fatigue tests should be run for each temperature to adequately describe the relationship between extreme fiber strain and the number of load repetitions to fracture. The extreme fiber stress should be varied such that the resulting number of load repetitions to fracture ranges from 1,000 to 1,000,000.

f. Report and presentation of results. The report of flexural fatigue test results should include the following:

(1) Density of test specimens.

(2) Number of load repetitions to fracture, N_f.

(3) Specimen temperature.

(4) Extreme fiber stress, σ.

The flexural fatigue relationship is plotted in figure 11-2.

11-2. Provisional fatigue data for bituminous concrete.

Use of the graph shown in figure 11-3 to determine a limiting strain value for bituminous concrete involves first determining a value for the elastic modulus of the bituminous concrete. Using this value and the design pavement service life in terms of load repetitions, the limiting tensile strain in the bituminous concrete can be read from the ordinate of the graph.

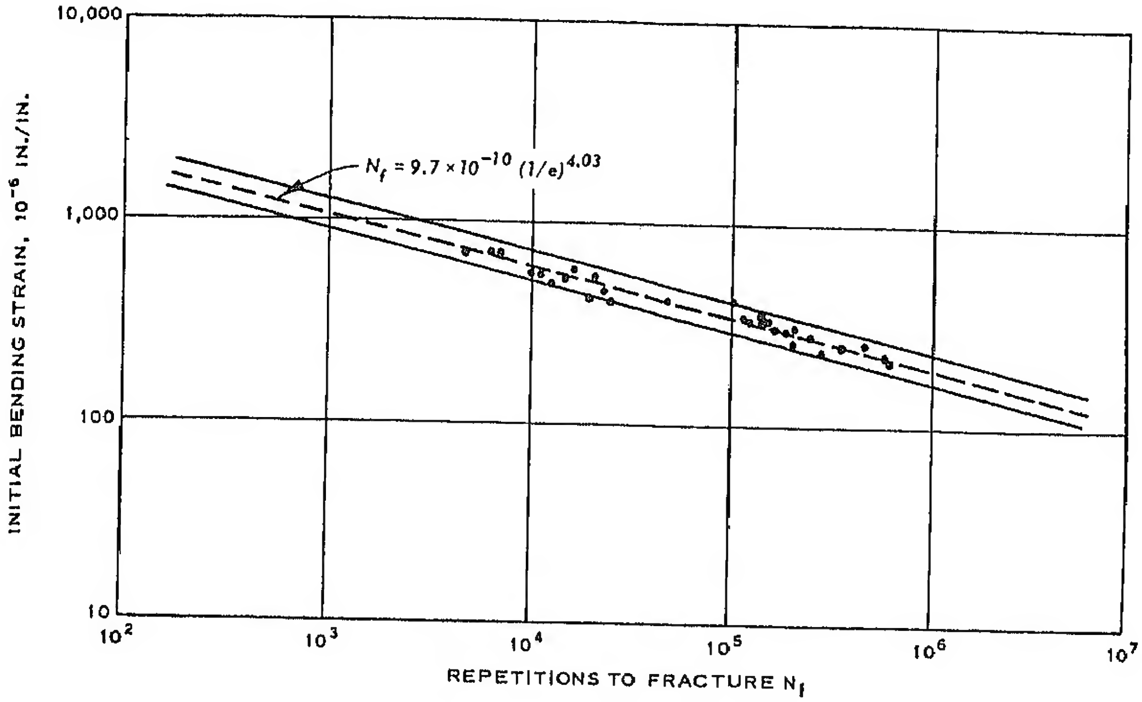


Figure 11-2. Initial mixture bending strain versus repetitions to fracture in controlled stress tests.

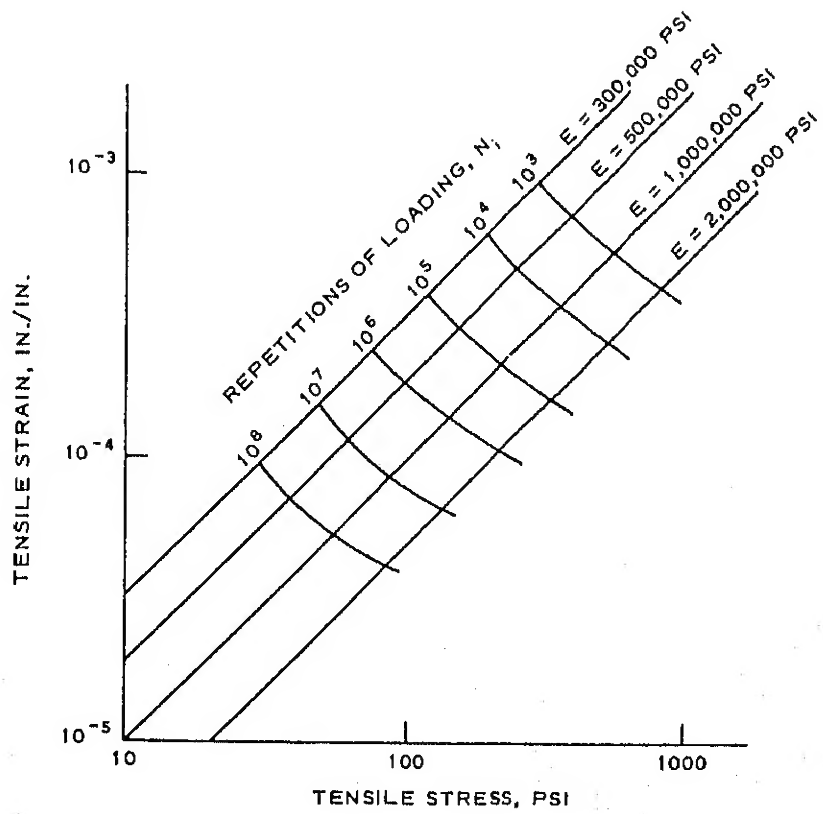


Figure 11-3. Provisional fatigue data for bituminous base course materials.

APPENDIX A

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16 Race Street, Philadelphia, PA 19103

617	Capping Cylindrical Concrete Specimens
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1559	Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus
1560	Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus
1561	Preparation of Bituminous Mixture Test Specimens by Means of California Kneading Compactor
1632	Making and Curing Soil-Cement Compression and Flexure Test Specimens in the Laboratory
1633	Compressive Strength of Molded Soil-Cement Cylinders
3202	Recommended Practice for Preparation of Bituminous Mixture Beam Specimens by Means of the California Kneading Compactor
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"Dynamic Testing as a Means of Controlling Pavements During and After Construction" by W. Heukelom and A. J. G. Klomp.

University of Illinois, Urbana, IL 61801

Report No. UILU-ENG-76-2009	Final Report, Data Summary, Resilient Properties of Subgrade Soil.

APPENDIX B

DESIGN EXAMPLES

B-1. Example problem.

To illustrate the application of the design procedure, consider a design for an Army Class III airfield. The airfield is located in Shreveport, Louisiana, and the subgrade is a lean clay classified CL. The design is to be for 200,000 passes of the C-130 aircracft. From TM 5-803-4, the design loading for the C-130 on the taxiway is found to be 155,000 pounds with a tire contact area of 400 square inches. For the runway interior the loading is reduced to account for aircraft lift and is considered to be 75 percent of loading for the taxiway. The reduction is accomplished by reducing the contact area, giving a contact area of 300 square inches. The design process may best be illustrated in steps. The basic steps are material investigation, determination of trial pavement sections, computation of critical strains, determination of applied strain repetitions, and computation of damage factors.

B-2. Step 1 — material investigation.

a. The evaluation of the subgrade is to be accomplished by field and laboratory studies. The subgrade is to be classified according to different material types and material processing. For this example, it is assumed that the subgrade is fairly uniform and consists of a compacted lean clay placed according to existing compaction requirements. The subgrade evaluation involves conducting a series of resilient modulus tests according to the procedures given in chapter 5. For a location such as Shreveport, Louisiana, it must be assumed that the subgrade would become saturated and thus the resilient modulus tests are conducted on saturated samples. A minimum of six samples should be tested and a design modulus determined for each sample. For determination of a design modulus, the data from the laboratory tests are plotted on a log-log plot of M_R versus σ_d and overlaid on the design curves as shown in figure B-1.

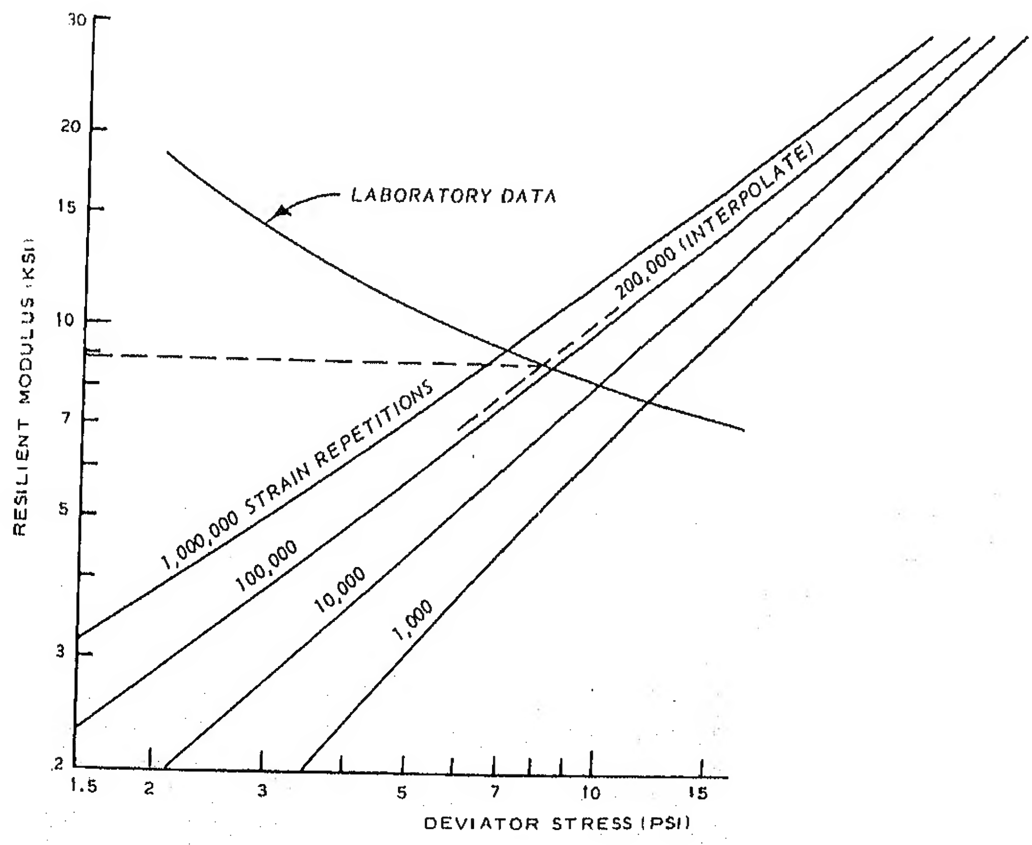

Figure B-1. Estimation of resilient modulus

For the design example, the design modulus obtained using this process is assumed to be 9,000 psi for both taxiway and runway designs. Base and subbase materials must be obtained that meet the requirements of Section 6 of TM 5-825-2, and AFM 88-6, Chapter 2. The modulus values for the base and subbase are to be determined by the procedures given in chapter 6 herein. Because the modulus of these materials depends on layer thicknesses, the modulus cannot be obtained until the trial sections are determined.

b. The bituminous surfacing must meet the requirements of table 6-4 of TM 5-825-2 as to minimum thickness and Section 7 of TM 5-825-2, and AFM 88-6, Chapter 2 as to composition. The modulus-temperature relationship determined according to the test procedures given in chapter 9 herein or by the provisional procedure given in chapter 10 herein. Assume for the example problem that the relationship as shown in figure B-2 is obtained from the test laboratory data. (For simplicity, these data will be used for both taxiway and runway.) From the climatic data, the design air temperature is obtained and the design modulus values are determined as shown in tables B-1 and B-2. To reduce the number of computations, the 12 groups are reduced to four groups as shown in table B-3. The Poisson's ratio for all materials are selected from the table given in paragrapha 2-3*b*.

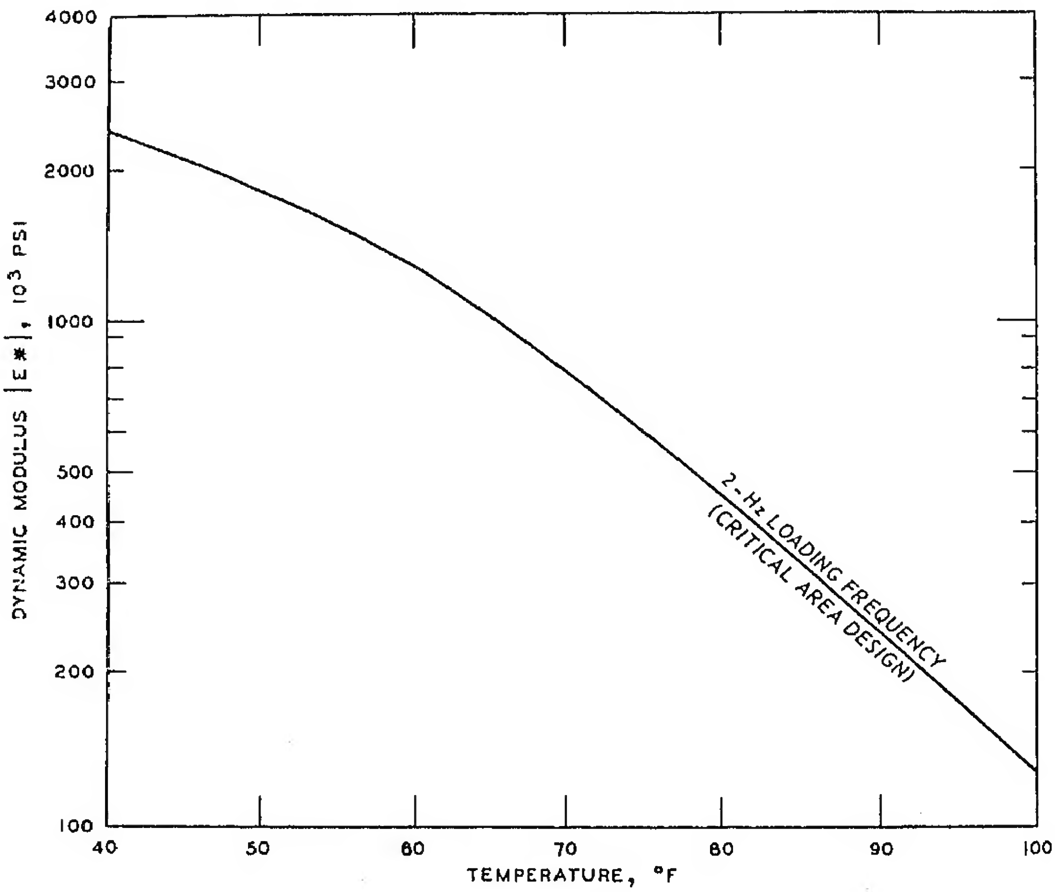

Figure B-2. Results of laboratory tests for dynamic modulus of bituminous concrete.

Table B-1. Bituminous concrete moduli for each month for conventional flexible pavement design based on subgrade strain

Month (1)	Average Daily Mean Air Temperature[a] degrees F. (2)	Average Daily Maximum Air Temperature[a] degrees F. (3)	Design Air Temperature[b] degrees F. (4)	Design Pavement Temperature[c] degrees F. (5)	Dynamic Modulus[d] |E*| 10^3 psi (6)
Jan	47.5	56.4	52	60	1270
Feb	50.7	60.1	55	64	1060
Mar	58.0	68.0	63	72	700
Apr	66.1	76.0	71	81	420
May	73.3	83.2	78	90	250
Jun	80.5	90.4	85	97	160
Jul	83.1	92.9	88	100	130
Aug	82.7	92.8	88	100	130
Sep	77.3	87.4	82	94	190
Oct	67.2	78.1	73	83	380
Nov	56.2	66.4	61	71	720
Dec	49.3	58.3	54	61	1200

[a] Determined from local climatological data for Shreveport, Louisiana.

[b] Average of values from columns 2 and 3.

[c] Estimated from 5-inch bituminous concrete thickness curve in figure 2-1. (Figure 2-1 is entered with the appropriate design air temperature.)

[d] Determined by laboratory testing of bituminous concrete.

Table B-2. Bituminous concrete moduli for each month for conventional flexible pavement design based on bituminous concrete strain

Month	Average Daily Mean Air Temperature[a] degrees F.	Design Pavement Temperature[b] degrees F.	Dynamic modulus |E*| 10^3 psi
Jan	47.5	56	1500
Feb	50.7	60	1270
Mar	58.0	67	920
Apr	66.1	76	570
May	73.3	84	360
Jun	80.5	92	220
Jul	83.1	95	180
Aug	82.7	95	180
Sep	77.3	89	260
Oct	67.2	77	540
Nov	56.2	65	1000
Dec	49.3	57	1400

[a] Determined from local climatological data for Shreveport, Louisiana.

[b] Estimated from 5-inch bituminous concrete thickness curve in figure 2-1. (In design for bituminous concrete strain, the average daily mean air temperature is used as the design air temperature for entering figure 2-1.)

Table B-3. Grouping traffic into traffic groups according to similar asphalt moduli

		Modulus Values, kips per square inch				
		For Computation of Asphalt Damage		For Computation of Subgrade Damage		
Group	Month	Monthly Values	Group Average	Monthly Values	Group Average	Percent of Total Traffic
1	Jan	1500	1390	1270	1180	25.0
	Dec	1400		1200		
	Feb	1270		1060		
2	Nov	1000	960	720	710	16.7
	Mar	920		700		
3	Apr	570	490	420	400	25.0
	Oct	540		380		
	May	360		250		
4	Sep	260	210	190	150	33.3
	Jun	220		160		
	Jul	180		130		
	Aug	180		130		

B-3. Step 2 — determination of initial section.

a. For determining the initial section, the subgrade CBR is estimated by converting the modulus-temperature relationship is determined according to the test procedures given in chapter 9

b. From the design for Army Class III airfield (fig. 8-3 of the TM 5-825-2), the total thickness of pavement required for a gross aircraft load of 155,000 pounds, 200,000 passes and a subgrade CBR of 6 is determined to be approximately 28 inches. For the runway design, the thickness would be determined based on a gross aircraft load of 116,250 pounds and would result in an estimated thickness of 24 inches. For taxiway design, subgrade damage factors will be computed for pavement thicknesses of 27, 30, and 33 inches in an attempt to bracket the final required pavement thickness. The total thickness of pavement is made up of the asphalt thickness, base thickness, and subbase thickness. The section for a thickness of 30 inches is shown in figure B-3.

c. For runway design, thicknesses of 20, 24, and 26 inches are assumed for the initial sections for computing the subgrade damage factor. The section for 24 inches is shown in figure B-4. In the initial section, a 5-inch asphalt layer is assumed for the taxiway design, and a 4-inch asphalt layer is assumed for the runway design. After determining the total thickness required for these asphalt thicknesses, the design can be refined for other asphalt thicknesses.

B-4. Step 3 — computation of strains.

a. The strain at the bottom of the asphalt layer and the strain at the top of the subgrade are computed for each traffic grouping shown in table B-3. The data needed for input into the BISAR computer program are given in the input guide listed in table B-4. The specific input in the computation of subgrade strains for a taxiway design for the pavement section shown in figure B-12 is given in table B-5. Note that the input contains data for one run, but four runs would be required to compute the subgrade data, i.e., one run for each grouping to account for variation in asphalt modulus. The strain is computed considering only two of the four main gears; the transverse

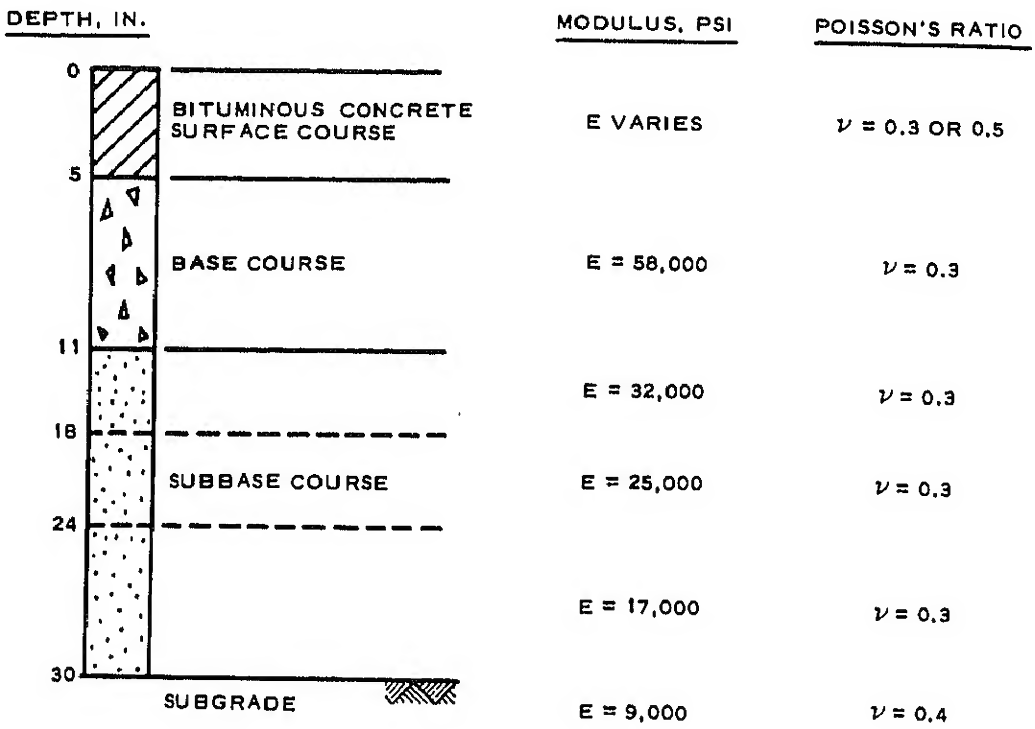

Figure B-3. Section for pavement thickness of 30 inches for initial taxiway design.

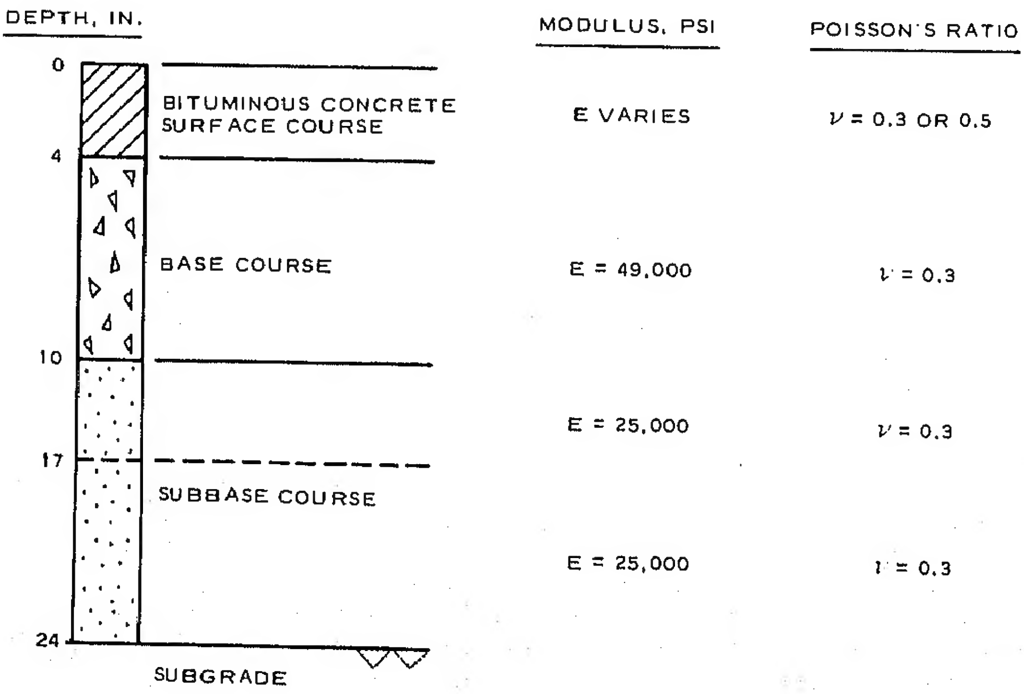

Figure B-4. Section for pavement thickness of 24 inches for initial runway design.

spacing of the tires is sufficiently large to prevent an overlapping effect for the other two tires. The individual tire loading is computed by considering 90 percent of the gross load as on the main gear, equally distributed between the four tires of the main gear, resulting in a design tire loading of 34,875 pounds. The radius of the loaded area is computed as a circle having an equal area for the tire contact area. A contact area of 400 square inches results in a radius of contact area of 11.25 inches. The pavement system is a six-layer system having full friction between layers; thus, the value of reduced interface compliance (ALK) is 0. For a flexible pavement system, the rough computational procedure is sufficiently accurate. The subgrade strain is computed at the top of the subgrade layer and under the center of one of the tires and midway between the tires. The maximum strain is found to occur under the tire. The data coded for input into the BISAR computer program for computing subgrade strain is shown in table B-5.

b. A similar set of runs is made for the computation of the strain at the bottom of the asphalt layer. This set of runs will use the asphalt moduli determined for consideration of asphalt strains and given in table B-3. For computing the strains for the runway design, the load and contact area are reduced by 75 percent. The resulting tire loading is 26,125.25 pounds applied over a circular contact area having a radius of 9.77 inches.

B-5. Step 4 — determination of applied repetitions.

a. The design is for 200,000 passes of the aircraft over the life of the pavement. The pavement life has been divided into four periods as shown in table B-3. Considering that the traffic is to be equally distributed throughout the year would result in 25, 16.7, 25, and 33.3 percent of the traffic to be applied in the first, second, third, and fourth period, respectively.

b. The 200,000 passes will result in a total number of effective strain repetitions that will be a function of transverse location on the pavement and on the depth at which the strain is being considered. From plots showing the conversion from passes to strain repetitions, figures 4-1 through 4-30 for the taxiway and runway, the conversion percentages are determined. For the taxiway and depth to the top of subgrade of 30 inches, the maximum conversion percentage for converting passes to effective strain repetitions is approximately 100. This maximum occurs at a distance of 86 inches from the centerline of the taxiway. Thus, the effective number of subgrade repetitions would be 200,000. For consideration of the asphalt strain at a depth of 5 inches, the conversion percentage is approximately 50, resulting in 100,000 strain repetitions. The conversion percentages for the runway are 60 and 30 for consideration of subgrade strain and asphalt strain, respectively.

c. The effective number of strain repetitions for a traffic group then is determined by multiplying the total strain repetitions by the factor of traffic occurring in a group.

B-6. Step 5 — computation of damage factors.

a. The damage factor for one traffic group is defined as n/N where n represents the effective strain repetitions for that group and N equals the allowable numbers of strain repetitions as computed from equations 2-3 and 2-4. The damage factors for the different periods are summed to obtain the cumulative damage factor.

b. The computations were performed by use of the computer programs SUBGRAD for the subgrade damage and ASPHALT for the asphalt damage. The data file, SDATA1, required by the program SUBGRAD for computing the subgrade damage factor for 33-, 30-, and 27-inch pavements is given in table B-6 and the output is given in table B-7. The data file ADATA1 required by the program ASPHALT for computing the asphalt damage factor for the asphalt is given in table B-8 and for the output in table B-9.

c. To speed the design procedure, the subgrade damage factor was computed for several pavement thicknesses and the results for the taxiway pavement plotted as shown in figure B-5. For pavements having an asphalt concrete thickness of 5 inches, the subgrade damage factor was computed for thicknesses of 27, 30, and 33 inches. From the plot of damage factor versus pavement thickness, it is determined that the required thickness for the taxiway pavement would be 28.8 inches, which should be rounded to 29 inches. Using the 29-inch overall thickness as a constant thickness, the subgrade damage factor can be computed for varying thicknesses of asphalt concrete. Lines can then be constructed that will provide the total thickness for each asphalt concrete thickness. Alternate designs, rounded to the nearest inch, might be 6-inch asphalt concrete with 28 inches in total thickness or 4-inch asphalt concrete with 30 inches in total thickness. The relationship between pavement thickness and subgrade damage factors for the runway is given in figure B-6.

d. For these designs, the asphalt damage must be computed. Plots of asphalt damage versus asphalt thickness for both the taxiway and runway are

Table B-4. Input guide

TEXT	20A4

[A] NSYS: I 2

Columns 1-2 NSYS = number of problems

[B] NLAYS: I 2, iSMO: I 3, iRED: I 1

Columns 1-2 NLAYS = number of layers in pavement system
Columns 3-5 iSMO = 0 request for rough computational procedure
iSMO = 1 request for smooth computational procedure (see Note 1)
Column 6 iRED = 0 AK(i) is input in [C] or
iRED = 1 ALK(i) is input in [C]

(Continued)

Note 1: The smooth calculation procedure is more stable but less efficient than the rough procedure and is used for systems with frictionless slip between the layers or for cases where numerical instabilities are expected.

Table B-4. (Continued)

TEXT	20A4

[D]

NLOAD
I 2

Columns 1-2 NLOAD = number of loaded areas

Load Information: One card for each load

[E]

Columns 1-10 LDSTRS(i) = vertical load in units of load for loaded area i
Columns 11-20 RADIUS(i) = radius of loaded area i
Columns 21-30 X(i) = abscissa of center of loaded area
Columns 31-40 Y(i) = ordinate of center of loaded area
Columns 41-50 HOSTR(i) = horizontal load in units of load for loaded area i
Columns 51-60 PSI(i) = angle of HOSTR(i) with respect to positive X-axis in degrees

LDSTRS	RADIUS	X	Y	HOSTR	PSI
F10.0	F10.0	F10.0	F10.0	F10.0	F10.0

(Continued)

Table B-4. (Continued)

TEXT	20A4

[C]

Layer Information: One card for each layer (see Note 2)

```
Columns 1-10       E(i) = moduli of layer i
Columns 11-20     NU(i) = Poisson's ratio
Columns 21-30  Thick(i) = thickness of layer i
Columns 31-40     AK(i) = interface compliance
           or    ALK(i) = reduced interface compliance (see Note 3)
```

Cards for layers i=1 thru NLAYS - 1

E(i)	NU(i)	THICK(i)	AKI(i) or ALK(i)
F10.0	F10.0	F10.0	F10.0

Card for layer i = NLAYS

E(i)	NU(i)
F10.0	F10.0

(Continued)

Note 2: Columns 21-40 left blank for last layer

Note 3: AK(i) values are generally very small thus it may be more

desirable to use ALK(i) where ALK(i) = $\frac{E_i}{1 + V_i}$ AK(i)

for complete adhesion between layers i and i + 1 set AK(i) = ALK(i) = 0

for almost frictionless slip between layers set = $\frac{E_i}{1 + V_i}$ AK(i) = ALK(i) $\geq$ 1000

Table B-4. (Concluded)

TEXT	20A4

[F]

NPOS
I 2

Columns 1-2 NPOS = the number of position for which stress, strains, and displacements are to be computed

[G]

Position data; one card for each position

Columns 1-2 LAYER(i) = layer number for position i
Columns 11-20 AX(i) = abscissa of position
Columns 21-30 AY(i) = ordinate of position
Columns 31-40 DEPTH(i) = depth from pavement surface to position
Columns 41-50 ETA(i) = angle from which position is observed with respect to the direction of the tangential loading

LAYER	AX	AY	DEPTH	ETA
I 2	F 1 0 . 0	F 1 0 . 0	F 1 0 . 0	F 1 0 . 0

If another problem is needed return to [B] and repeat input through [G].

Table B-5. Data input to BISAR computer program

```
EXAMPLE INPUT FOR COMPUTATION OF SUBGRADE STRAINS t = 30 IN.
 1
 6  0 1
1 1 8 0 0 0 0 .   . 3         5 .      0 .
5 8 0 0 0 .       . 3         6 .      0 .
3 2 0 0 0 .       . 3         7 .      0 .
2 5 0 0 0 .       . 3         6 .      0 .
1 7 0 0 0 .       . 3         6 .      0 .
9 0 0 0 .         . 4         6 .      0 .
 2
3 4 8 7 5 .       1 1 . 2 8   0 .      0 .      0 .      0 .

3 4 8 7 5 .       1 1 . 2 8   0 .      6 0 .    0 .      0 .
 1
 6                0 .         0 .      3 0 .    0 .
```

Table B-6. Data file for computing subgrade damage for pavement thicknesses of 33, 30, and 27 inches

```
List SDATA1

100    Taxiway design subgrade damage thickness = 33 inches
110    4 200000
120    .25  .1666667  .25  .333333
130    9000.  9000.  9000.  9000.
140    .000654  .000698  .000741  .00806
150    Taxiway design subgrade damage thickness = 30 inches
160    4  200000
170    .25  .1666667  .25  .333333
180    9000.   9000.  9000.  9000.
190    .000733,  .000789  .000244.  000927
200    Taxiway design subgrade damage thickness = 27 inches
210    4  200000
220    .25  .166667  .25  .33333
230    9000.  9000.  9000.  9000.
240    .000831  .000908  .000980  .001080
250    End of data
260    0  0
```

Table B-7. Program output for subgrade damage for pavement thicknesses of 33, 3
and 27 inches

* Run SUBGRAD

00 Taxiway Design Subgrade Damage Thickness = 33 inches

Damage based on subgrade strain criteria

Sub Modulus	Subg Strain	Allow Reps	Applied Reps	Damage
9000.	0.000654	7523438.	50000.	0.665-02
9000.	0.000698	3758888.	33333.	0.895-02
9000.	0.000741	1981678.	50000.	0.258-01
9000.	0.000806	807168.	66667.	0.835-01

Total Damage = 0.1235E 00

50 Taxiway Design Subgrade Damage Thickness = 30 inches

Damage based on subgrade strain criteria

Sub Modulus	Subg Strain	Allow Reps	Applied Reps	Damage
9000.	0.000733	8885301.	50000.	0.825-01
9000.	0.000789	1018577.	33333.	0.338-01
9000.	0.000844	493454.	50000.	0.10E 00
9000.	0.000927	181174.	66667.	0.37E 00

Total Damage = 0.5256E 00

20 Taxiway Design Subgrade Damage Thickness = 27 inches

Damage based on subgrade strain criteria

Sub Modulus	Subg Strain	Allow Reps	Applied Reps	Damage
9000.	0.000831	582449.	50000.	0.86E-01
9000.	0.000908	231418.	33333.	0.14E 00
9000.	0.000980	100039.	50000.	0.50E 00
9000.	0.001080	32112.	66666.	0.21E 00

Total Damage = 0.281E 01

Table B-8. Data file for computing asphalt damage

```
List ADATA1

100      Taxiway Design; A5.  Thickness = 4 inches
110      4 100000
120      .25  .16667  .25  .33333
130      1390000.  860000.  490000.  210000.
140      .000217  .000228  .000247  .000219
150      Taxiway Design; A5.  Thickness = 5 inches
160      4 100000
170      .25  .16667  .25  .33333
180      1390000.  960000.  490000.  210000.
190      .000218  .000234  .000267  .000263
200      Taxiway Design; A5.  Thickness = 6 inches
210      4 100000
220      .25  .166667  .25  .3333
230      1390000.  960000.  490000.  210000.
240      .000200  .000227  .000270  .000295
250      End of data
260      0  0
```

Table B-9. Program output for asphalt damage

* Run ASPHALT

00 Taxiway Design; A5. Thickness = 4 inches

Damage based on asphalt strain criteria
and on 100000. total strain repetitions.

Asp Modulus	Asph Strain	Allow Reps	Applied Reps	Damage
1390000.	0.000217	42328.	25000.	0.59E 00
960000.	0.000228	88688.	16687.	0.19E 00
490000.	0.000247	356568.	25000.	0.70E-01
810000.	0.000219	6333934.	33333.	0.54E-03

Total Damage = 0.854E 00

50 Taxiway Design; A5. Thickness = 5 inches

Damage based on asphalt strain criteria
and on 100000. total strain repetitions.

Asp Modulus	Asph Strain	Allow Reps	Applied Reps	Damage
1390000.	0.000218	47554.	25000.	0.52E 00
960000.	0.000234	77834.	16667.	0.21E 00
490000.	0.000267	241586.	25000.	0.10E 00
210000.	0.000263	2491763.	33333.	0.13E-01

Total Damage = 0.857E 00

200 Taxiway Design; A5. Thickness = 6 inches

Damage based on asphalt strain criteria
and on 100000. total strain repetitions.

Asp Modulus	Asph Strain	Allow Reps	Applied Reps	Damage
1390000.	0.000200.	63638.	25000.	0.39E 00
960000.	0.000227	90598.	16667.	0.18E 00
490000.	0.000270	228459.	85000.	0.11E 00
210000.	0.000295	1403381.	33333.	0.84E-01

Total Damage = 0.710E 00

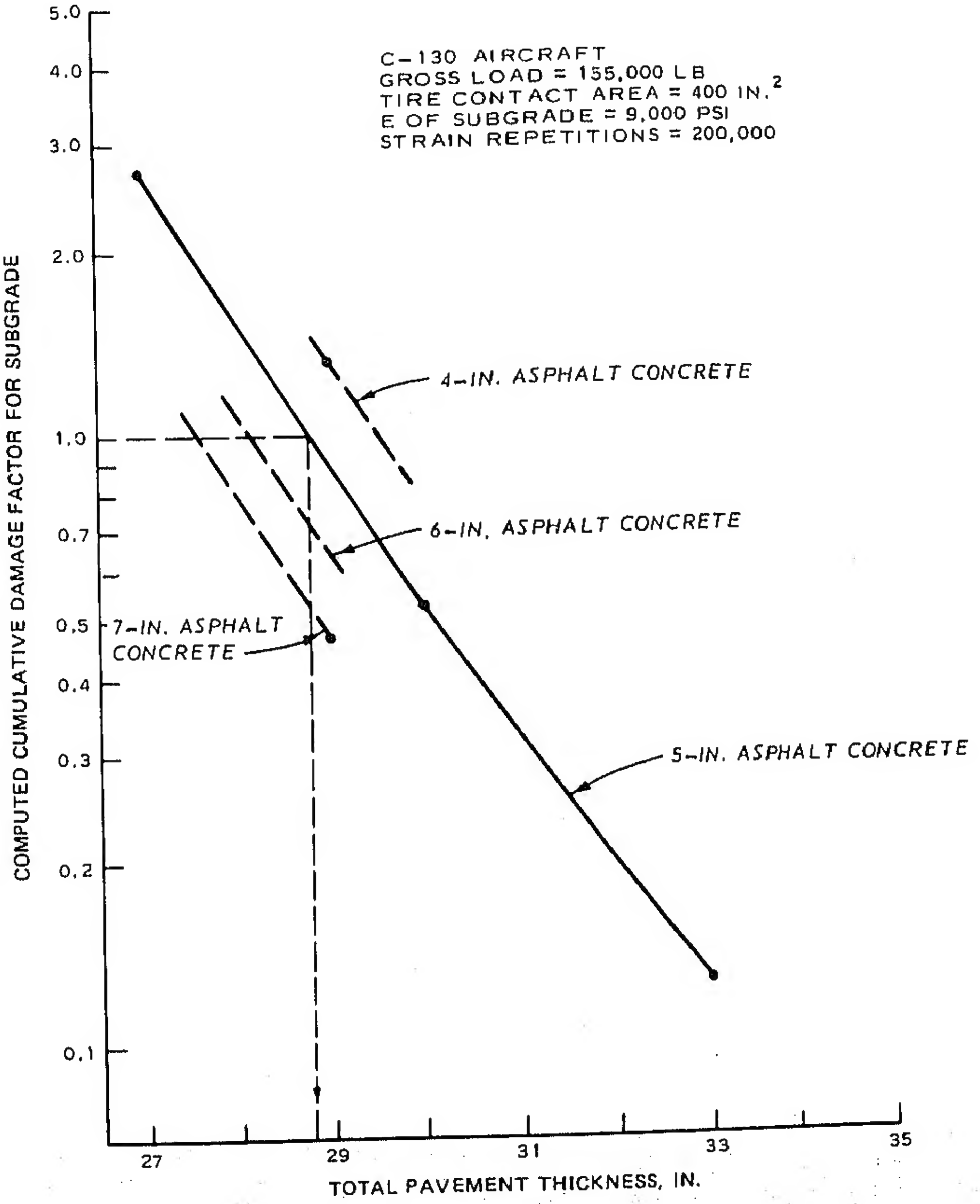

Figure B-5. Pavement design for taxiways.

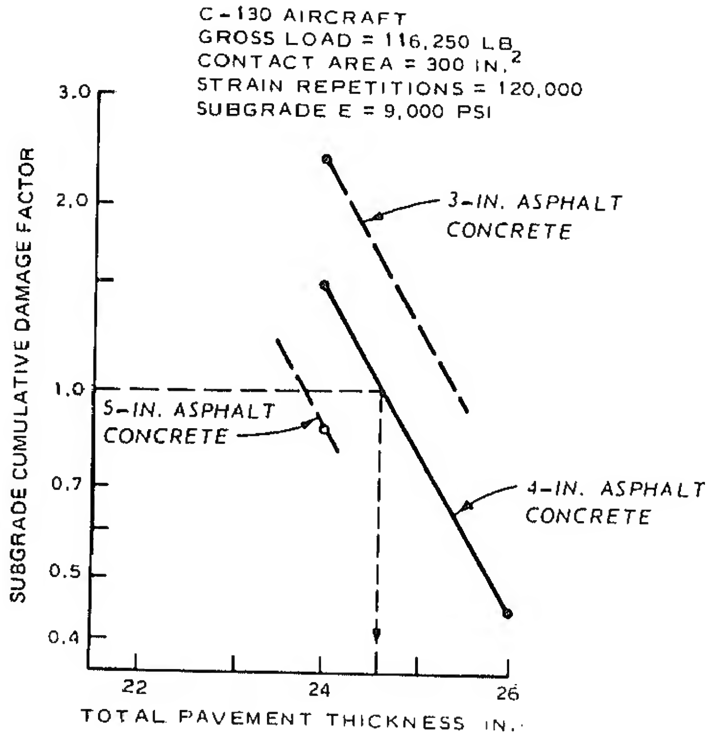

Figure B-6. Design for runways.

given in figure B-7. The asphalt damage factor would not control the asphalt thickness. The minimum thickness of the asphalt layer would be dictated by the minimum thickness criterion in the basic manual.

B-7. Design of ABC pavement.

a. The thickness of the ABC pavement required ...way design is estimated by considering ...ss of conventional pavement, i.e., 5 ...asphaltic concrete and 24 inches of ...ase and subbase. For this conventional the effective thickness would be 34 inches which when converted to an ABC pavement would give an estimated thickness of 17 inches (computed by using the equivalence of 2 for bound materials). For computation of the fatigue damage and subgrade damage monthly time periods are used as shown in tables B-10 and B-11, respectively. Normally for ABC design the subgrade damage will be the controlling criteria and thus the thickness for satisfying the subgrade criteria is first determined. The subgrade strains are computed for six time periods so as to produce a plot as shown in figure B-8. From this plot the subgrade strains for each time period are determined and

C-130 AIRCRAFT

	TAXIWAY	RUNWAY
GROSS LOAD, LB	155,000	116,250
CONTACT AREA, IN.^{2}	400	300
STRAIN REPETITIONS	100,000	60,000

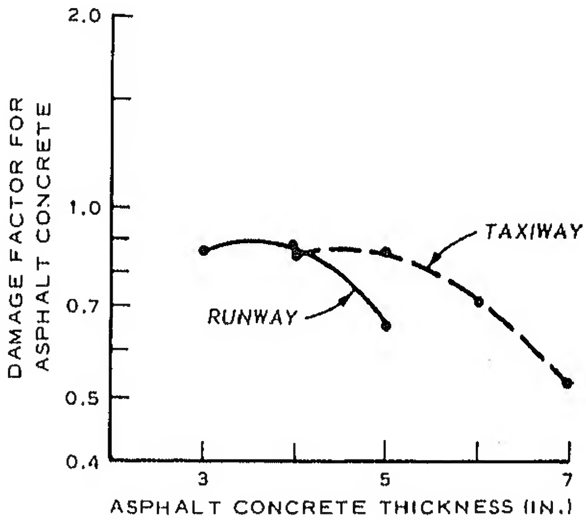

Figure B-7. Design for asphalt concrete surface.

are given in table B-12. The data shown in table B-12 are input into the computer program SUBGRAD to compute the subgrade damage factor. It is noted that an equivalent thickness of 34 inches is used to determine the applied strain repetitions, resulting in the same number of strain repetitions as was used for the design of the conventional pavement. Damage factors were computed for pavement thicknesses of 16, 17, 19, and 21 inches and from the data from which the plot of damage factor versus pavement thickness (fig B-9) was developed. From the plot the thickness for a damage factor of 1 is determined to be 16.7 inches which if rounded to the nearest inch would give a design thickness of 17 inches. The fatigue damage factor based on the asphalt criteria is then computed for a pavement thickness of 17 inches.

b. The plot of asphalt strain versus asphalt modulus is shown in figure B-10. The asphalt strain for each time period is given in table B-12. Using the computer program ASPHALT, the fatigue damage factor is computed to be 0.15, which is considerably less than 1. Thus a pavement thickness of 17 inches meets both the subgrade criteria and the asphalt fatigue criteria.

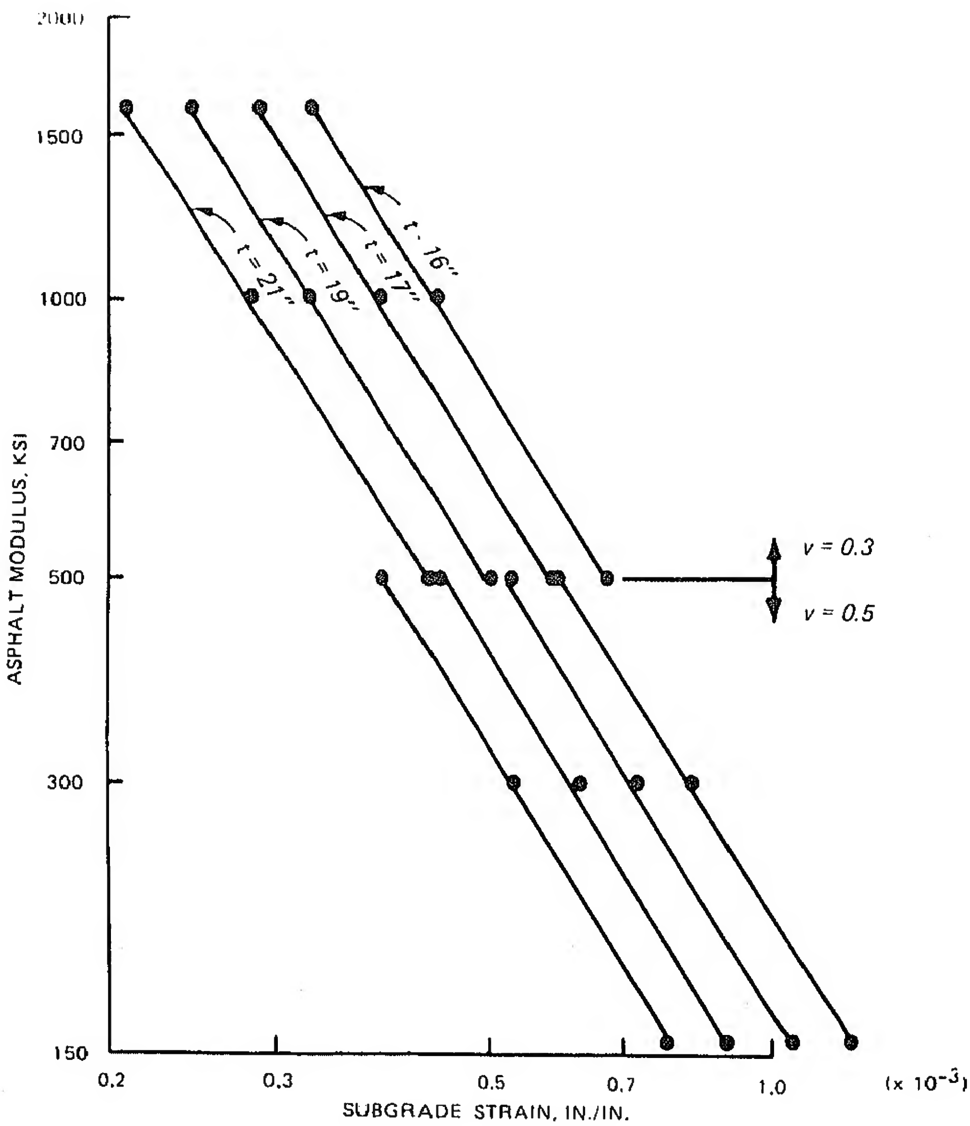

Figure B-8. Computed strain at the top of the subgrade for taxiway design.

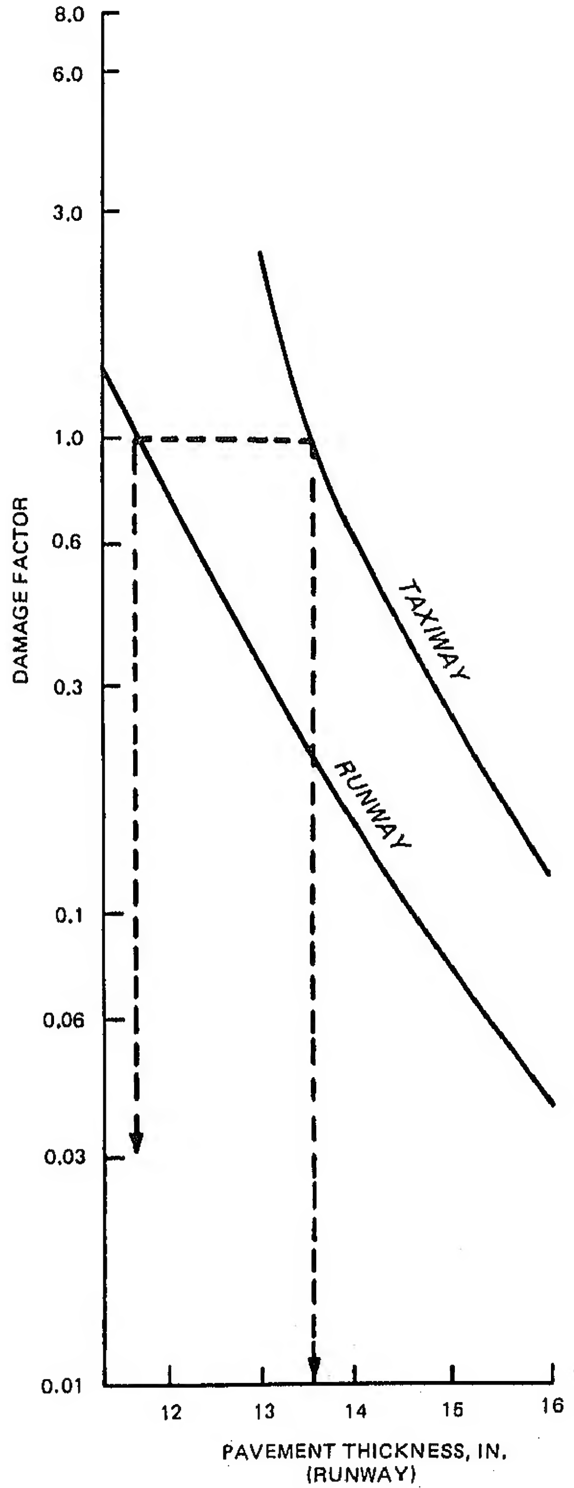

Figure B-9. Damage factor versus pavement thickness.

Table B-10. Bituminous concrete moduli for each month for ABC pavement design based bituminous concrete strain

Month	Average Daily Mean Air Temperature degrees F.	Design Pavement Temperature degrees F.	Dynamic Modulus |E*| 10^3 psi
Jan	47.5	54	1600
Feb	50.7	57	1400
Mar	58.0	64	1060
Apr	66.1	72	700
May	73.3	80	460
Jun	80.5	88	280
Jul	83.1	91	230
Aug	82.7	91	230
Sep	77.3	85	340
Oct	67.2	73	670
Nov	56.2	61	1200
Dec	49.3	56	1500

Table B-11. Bituminous concrete moduli for each month for ABC pavement design based on subgrade strain

Month	Average Daily Mean Air Temperature degrees F.	Average Daily Maximum Air Temperature degrees F.	Design Air Temperature degrees F.	Design Pavement Temperature degrees F.	Dynamic Modulus |E*| 10^3 psi
Jan	47.5	56.4	52	57	1400
Feb	50.7	60.1	55	62	1150
Mar	58.0	68.0	63	70	790
Apr	66.1	76.0	71	77	540
May	73.3	83.2	78	86	320
Jun	80.5	90.4	85	95	180
Jul	83.1	92.9	88	97	160
Aug	82.1	92.8	88	97	160
Sep	77.3	87.4	82	91	230
Oct	67.2	78.1	73	82	400
Nov	56.2	66.4	61	69	830
Dec	49.3	58.3	54	61	1200

Table B-12. Data for computing damage factors for taxiway design

Table B-12. Data for computing damage factors for taxiway design

Month	Strain Repetitions	Subgrade Modulus psi	Subgrade Strain, inches/inch × 10^{-5}				Asphalt Modulus kips per square inch	Asphalt Strain, inches/inch × 10^{-6}
			t = 16 inches	t = 17 inches	t = 19 inches	t = 21 inches		t = 17 inches
Jan	16,666	9,000	35	31	26	23	1600	87
Feb	16,666	9,000	40	35	30	26	1400	96
Mar	16,666	9,000	50	44	37	32	1060	118
Apr	16,666	9,000	64	56	47	41	700	161
May	16,666	9,000	78	68	59	50	460	194
Jun	16,666	9,000	112	94	82	74	280	275
Jul	16,666	9,000	120	104	89	78	230	315
Aug	16,666	9,000	120	104	89	78	230	315
Sep	16,666	9,000	96	84	72	62	340	240
Oct	16,666	9,000	68	60	53	48	670	167
Nov	16,666	9,000	49	43	37	31	1200	108
Dec	16,666	9,000	39	34	29	25	1500	91

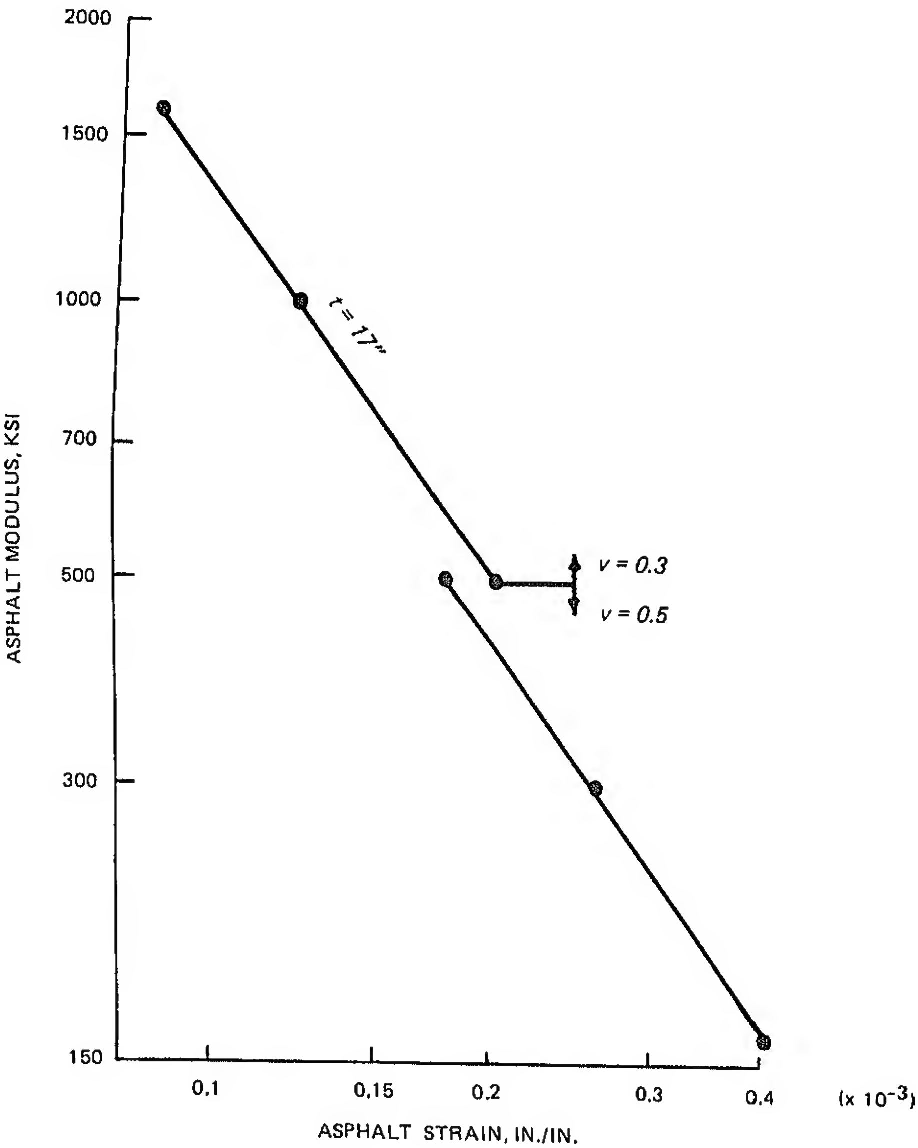

Figure B-10. Computed strain at the bottom of the asphalt for taxiway design.

c. The runway design is accomplished in the same manner as the taxiway design. The conventional runway section of 4 inches of asphaltic concrete and 25 inches of granular base and subbase converted to a 33-inch effective thickness. An ABC pavement of 14-1/2 inches would be required to give the same effective thickness. Based on the estimated thickness the subgrade damage factor was computed for pavement thicknesses of 13, 14, and 16 inches. The aircraft wheel load and the number of load repetitions for the computations were the same as used in the design for the conventional section. The subgrade strains and the asphalt strains as a function of pavement thickness are given in figures B-11 and B-12, respectively. The data for computing the damage factors are given in table B-13. The plot of the subgrade damage factor versus thickness is given in figure B-10. From the plot it is determined that a 13.5 inch ABC pavement would satisfy the subgrade criteria. The asphalt fatigue damage factor for a 13-inch pavement was computed to be 0.24, thus determining that the 13.5-inch pavement satisifies both criteria.

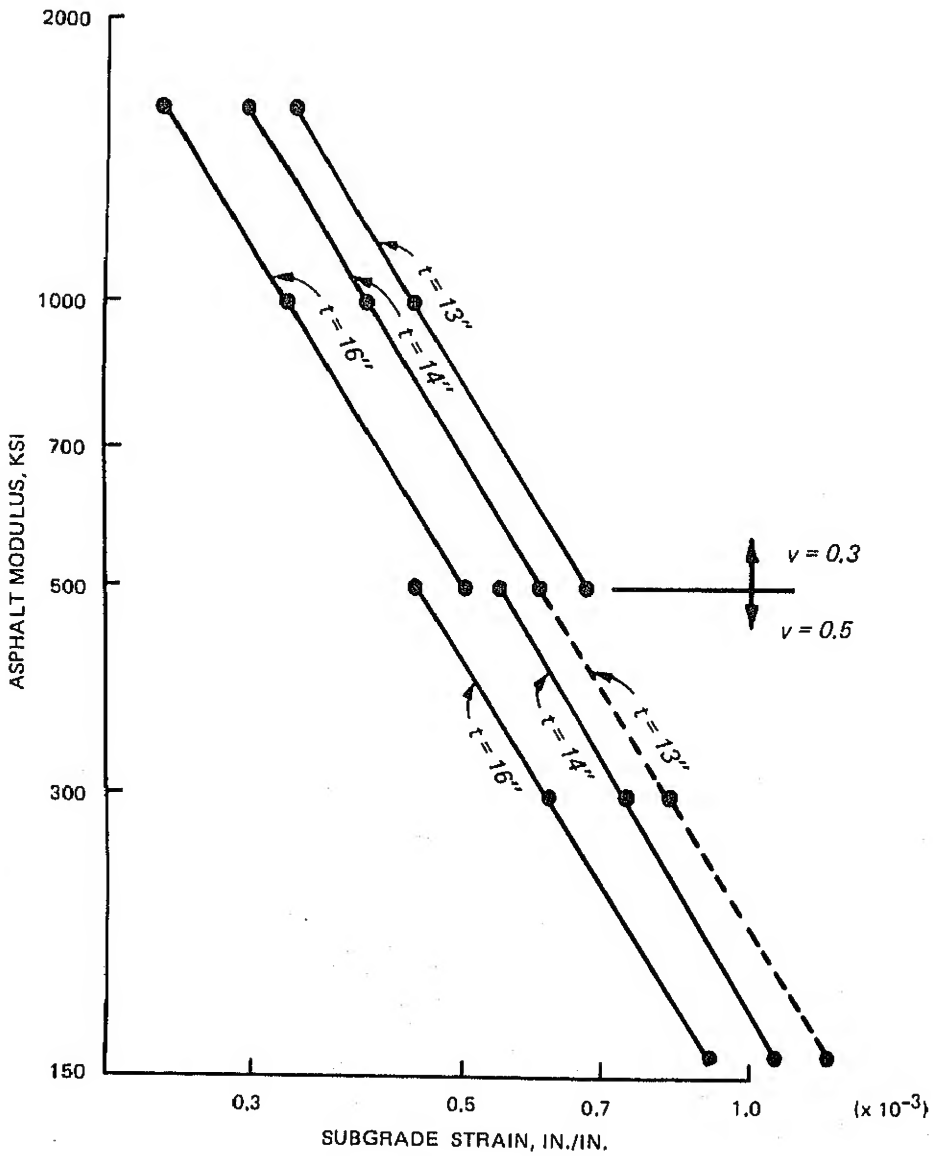

Figure B-11. Computed strain at the top of the subgrade for runway design.

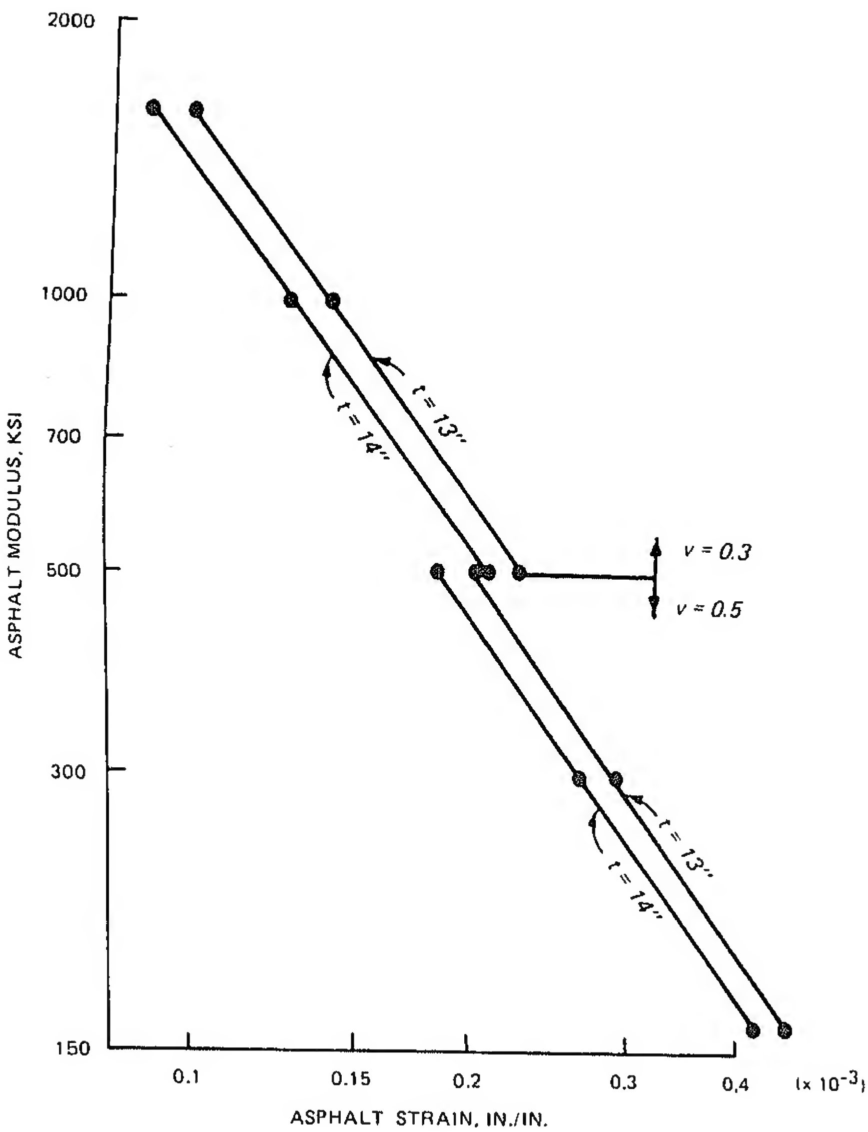

Figure B-12. Computed strain at the bottom of the asphalt for runway design.

Table B-13. Data for computing damage factors for runway design

Month	Strain Repetitions	Subgrade Modulus	Subgrade Strain, inches/inch × 10^{-5}			Asphalt Modulus, kips psi	Asphalt Strain, inches/inch × 10^{-6}	
			t = 13 inches	t = 14 inches	t = 16 inches		t = 13 inches	t = 14 inches
Jan	10,000	9,000	36	32	26	1600	99	91
Feb	10,000	9,000	40	36	30	1400	110	100
Mar	10,000	9,000	51	46	38	1060	134	123
Apr	10,000	9,000	64	58	48	700	183	167
May	10,000	9,000	79	71	59	460	220	200
Jun	10,000	9,000	113	100	84	280	310	285
Jul	10,000	9,000	121	106	91	230	357	325
Aug	10,000	9,000	121	106	91	230	357	325
Sep	10,000	9,000	97	86	73	340	372	248
Oct	10,000	9,000	69	62	51	670	190	172
Nov	10,000	9,000	49	45	37	1200	122	112
Dec	10,000	9,000	39	35	29	1500	105	95

APPENDIX C

COMPUTER PROGRAM SUBGRADE

```
LIST ASPHALT

010       DIMENSION  PCT(12), AS(12), STR(12)
020       CHARACTER TITLE*6(12)
030C
040C          ****VARIABLE LIST****
050C      NP IS THE NUMBER OF TRAFFIC GROUPS
060C      REPS IS THE TOTAL NUMBER OF STRAIN REPETITIONS
070C      LINENO IS A DUMMY VARIABLE TO STRIP LINE NUMBERS FROM
            DATA FILE
080C      PCT IS THE FRACTION OF TRAFFIC IN EACH TRAFFIC GROUP
090C      AS IS THE ASPHALT MODULUS FOR EACH TRAFFIC GROUP
100C      STR IS THE COMPUTER ASPHALT STRAIN FOR EACH TRAFFIC GROUP
110C      TITLE IS THE TITLE FOR THE DATA
120C             **********
130C      DATA FILE NAMED 'ADATA1' IS REQUIRED BEFORE EXECUTION OF
            PROGRAM
140C      LINE NUMBERS ARE REQUIRED
150C
160       CALL ATTACH(10,"/ADATA1:", 3, 0,,)
170       1 CONTINUE
180C
190C      THE FIRST LINE OF THE DATA FILE IS THE TITLE LINE
200C
210       READ (10,803) TITLE
220C      *************
230       803 FORMAT (12A6)
240C
250C      THE SECOND LINE IS THE NUMBER OF TRAFFIC GROUPS AND STRAIN
            REPS
260C      TO END DATA INPUT O'S FOR THIS LINE
270C
280       READ (10,900) LINENO, NP, REPS
290C      *************
300       900 FORMAT (V)
310       IF (NP,EQ.0) GO TO 1000
320C
330C      DATA LINE FOR INPUT OF THE TRAFFIC FRACTION FOR EACH
            TRAFFIC GROUP
340       READ (10,900)LINEO, (PCT(I), I=1, NP)
350C      *************
360C
```

(Continued)

```
C     DATA LINE FOR INPUT OF ASPHALT MODULUS FOR EACH TRAFFIC GROUP
C
      READ (10,900) LINENO, (AS(I), I = 1, NP)
C     *************
C
C     DATA LINE FOR INPUT OF THE ASPHALT STRAIN FOR TRAFFIC GROUP
C
      READ (10,900) LINENO, (STR(I), I = 1, NP)
C     *************
      WRITE (6,804)
      WRITE (6,803) (TITLE(I), I = 1, 12)
      WRITE (6,800) REPS
         TOTAL = 0.0
      DO 10 I = 1, NP
         APP = PCT(I)*REPS
         ALL = 2.68-5*ALOG10(STR(I))-2,665*ALOG10(AS(I))
         ALL = 10**ALL
         DAM=APP/ALL
         TOTAL = TOTAL + DAM
      WRITE (6,801)AS(I), STR(I), ALL, APP, DAM
      10 CONTINUE
      WRITE (6,802) TOTAL
      GO TO 1
      800 FORMAT (//, 5X, "DAMAGE BASED ON ASPHALT STRAIN CRITERIA",/,
&               5X,"AND ON ", F10.0," TOTAL STRAIN REPETITIONS"./,
&     4X, "ASP MODULUS  ASPH STRAIN  ALLOW REPS  APPLIED REPS  DAMAGE",/)
      801 FORMAT (5X, F10.0, 2X, F10.6, 2X, F10.0, E13.2)
      802 FORMAT (5X,"*********************",/,5X, "TOTAL DAMAGE = "E12.3)
      804 FORMAT(//, 15X."* * * * * * * *",/)
      1000 CONTINUE
      STOP
      END
```

APPENDIX D

COMPUTER PROGRAM ASPHALT

LIST SUBGRAD

```
010      DIMENSION PCT(12), AS(12), STR(12)
020      CHARACTER TITLE*6(12)
030C        PCT IS FRACTION OF TRAFFIC IN EACH TRAFFIC GROUP
040C        AS IS THE ASPHALT MODULUS FOR EACH TRAFFIC GROUP
050C        STR IS THE COMPUTED ASPHALT STRAIN FOR EACH TRAFFIC
              GROUP
060C        NP IS THE NUMBER OF TRAFFIC GROUPS
070C        REPS IS THE TOTAL NUMBER OF EFFECTIVE STRAIN REPETITIONS
080C        LINENO IS A DUMMY VARIABLE FOR STRIPPING LINE NUMBERS
              FROM FILE
090C
100C        DATA FILE NAMED "SDATA1" REQUIRED BEFORE EXECUTION OF
              PROGRAM
110C         NOTE THAT LINE NUMBERS ARE REQUIRED ON THE DATA FILE
120C
130      CALL ATTACH (10,"/SDATA1:", 3,0,,)
140      1 CONTINUE
150C
160C     THE FIRST LINE OF THE DATA FILE IS A TITLE LINE
170C     THIS LINE IS ALSO INPUT FOR NEXT TO LAST LINE OF THE DATA
           FILE
180C
190      READ (10,803) TITLE
200C     *************
210      803 FORMAT (12A6)
220C
230C     DATA LINE FOR NUMBER OF TRAFFIC GROUPS AND TOTAL
           REPETITIONS
240C     FOR LAST LINE OF DATA FILE INPUT 0'S FOR EACH VARIABLE
250C
260      READ (10,900) LINENO, NP, REPS
270C     ***********
280      900 FORMAT (V)
290C
300C     IF THE VALUE OF NP IS 0 THE PROGRAM ENDS
310C
320         IF (NP, EQ, 0) GO TO 1000
330C
340C     DATA LINE FOR FRACTION OF TRAFFIC IN EACH TRAFFIC GROUP
350C
360      READ (10,900) LINENO, (PCT(I), I = 1, NP)
```

(Continued)

```
370C    ***********
380C
390C    DATA LINE FOR ASPHALT MODULUS FOR EACH TRAFFIC GROUP
400C
410     READ (10,900) LINENO, (AS(I), I = 1, NP)
420C    ***********
430C
440C    DATA LINE FOR ASPHALT STRAIN FOR EACH TRAFFIC GROUP
450C    THIS IS THE LAST FOR A PARTICULAR PROBLEM IF MORE DATA
460C    IS TO BE INPUT REPEAT LINES BEGINNING WITH TITLE LINE
470C    TO END PROGRAM INPUT TITLE LINE AND 0'S FOR NP AND REPS
480C
490     READ (10,900) LINENO, (STR(I), I = 1, NP)
500C    ***********
510     WRITE (6,804)
520     WRITE (6,803) (TITLE(I), I = 1, 12)
530     WRITE (6,800)
540          TOTAL = 0.0
550     DO 10 I = 1, NP
560        APP + PCT(I)*REPS
570        A = 000247 + 000245*ALOG10(AS(I))
580        B = 0658*(AS(I)**559)
590        ALL = 10000*(A/STR(I))**B
600        DAM = APP/ALL
610        TOTAL = TOTAL + DAM
620     WRITE (6,801) AS(I), STR(I), ALL, APP, DAM
630     10 CONTINUE
640       WRITE (6,802) TOTAL
650       GO TO 1
660     800 FORMAT (//, 5X, "DAMAGE BASED ON SUBGRADE STRAIN
          CRITERIA",/,
670     & 4X, "SUB MODULUS  SUBG STRAIN  ALLOW REPS  APPLIED REPS
          DAMAGE",/)
680     801 FORMAT (5X, F10.0, 2X, F10.6, 2X, F10.0, 2X, F10.0,
          E13.2)
690     802 FORMAT (5X," **********************",/, 5X, "TOTAL
          DAMAGE = "E12.3)
700     804 FORMAT (//, 15X," * * * * * * * *",/)
710     1000 CONTINUE
720     STOP
730     END
```

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